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Discussion of all papers is invited

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THE CHICAGO STOCK YARDS FIRE, MAY 19, 1934

By Loran D. Gayton

(Acting City Engineer, Chicago, Ill.)

Considerable difference of opinion still prevails as to the origin of fire. Even the engineering profession does not claim to know just when, where or how it first appeared upon this earth. The Greeks had an explanation for almost everything, and they did not overlook the phenomenon of fire. In their mythology we find the following story:

The secret of fire was in possession of Zeus, the god of thunder and lightning. The people of the earth desired very much to learn this secret, but Zeus was a cantankerous old individual and refused to divulge it. Fortunately there was another god, Prometheus by name, who was more socially inclined and had a friendly feeling toward the lowly dwellers upon the earth. The latter made known to Prometheus their desire to learn the secret of fire and the god promised that he would do his best to secure it. Therefore, on one dark and stormy night when Zeus was putting on his act, hurling bolts of lightning and thunder hither and yon, and raising "hell in the heavens" generally, Prometheus stole the secret of fire, brought it down to earth, and made it available to mankind.

Surely the engineering profession should be grateful to their ancient friend, Prometheus, for they have made use of his gift in many

ways. There is no doubt that fire has had a great part in shaping our present civilization and in bringing about our present day standards of living. Fire has been a servant to mankind and of tremendous benefit whenever it has been kept under control, but it is like a savage animal in leash, every now and then breaking loose and wreaking its vengeance upon us.

One of the more recent and spectacular attempts of the Fire-God to prove that he is not as yet completely subdued and that he is an ever present menace that must be reckoned with was at the Chicago Stock Yards on the afternoon of Saturday, May 19, 1934. In this instance, before he was finally driven back and subdued, an area of approximately 140 acres had been laid waste, and property to the value of $6,000,000 had been destroyed.

As a foreword to a description of the Chicago Stock Yards fire, may I quote Mr. W. C. Mabee, Chief Engineer of the Indianapolis Water Company, who wrote as follows in The Journal of June, 1934:

"The part played in the extinguishment of fires by the water department is all too often overlooked and generally goes unnoticed; yet a little contemplation of the subject will convince you that after all you cannot accomplish much in fire extinguishment without water and plenty of it.

"Just as an army without ammunition is helpless, so a fire department without adequate water supply is likewise helpless. Water is the ammunition with which all fires of consequence are fought, and on the proper utilization of that ammunition depends our fire fighting effectiveness. Fire Chiefs understand this but the public at large apparently fails to appreciate the full value of the water supply in fire emergencies. There is no glamour connected with it; no clanging of bells nor shrieking of sirens heralding the arrival of the ammunition at the scene, for it is already there waiting for the apparatus and the firemen to arrive. When, however, a fire occurs beyond the limits of the water mains, and results in a total loss, attention is forcefully drawn to the situation where the firemen were practically helpless from the lack of water.

"Water under pressure in the mains surrounding the scene of a potential fire is the silent watchman, ever on the job, ready to go into instant action with turning of a hydrant key or the melting of a sprinkler head fuse. It is the first line of defense in an incipient blaze—it extinguishes the last spark of a conflagration."

THE STOCK YARDS

The so-called Stock Yards, which is the property of the Union Stock Yards and Transit Company, extends from 39th Street on the north to 47th Street on the south and from Racine Avenue on the west to Halsted Street on the east. This area is, in general, covered with cattle pens and runways and various buildings necessary for

the receiving, feeding and handling of live stock. It is in this area that the fire occurred. It did not spread into the packing house district west of Racine Avenue.

The entire stock yards is privately owned, with no public streets or public water supply within its boundaries. The Stock Yards Company has a 3½ million gallon reservoir on its property just north of 41st Street. The City delivers water into this reservoir and from here it is pumped by the Stock Yards Pumping Station through a system of small diameter pipes.

In the northern part of the Stock Yards section, there are several groups of mainly double-deck wooden, sheep and hog pens of varied large areas. These groups are separated by railroad tracks. In the vicinity of Halsted and Exchange Avenue, there are brick sales stables, a large exhibition hall, a fire-proof hotel, and a few mercantiles. In the southern part, a frame car shed, and an individual packing plant are fairly well separated from the remainder of the district. The central portion of this section is occupied by open frame cattle pens.

In the Stock Yards and Packingtown district as a whole, the predominance of ordinary and frame construction in excessive areas, numerous unprotected floor openings, hazardous nature of occupancies and serious congestion, combine to create an essentially high potential hazard.

The Bureau of Engineering of the Department of Public Works has long regarded the Stock Yards district as the greatest fire hazard in the City of Chicago. Over a long period of years, the Bureau has made plans and has laid down structures costing millions of dollars in order to provide the protection which it felt sure would some day be required. In all its planning and building, the Bureau had in mind the safeguarding of not only the Stock Yards area itself, but also the lives and property of the people in the surrounding territory.

The Stock Yards area is the privately owned property of the Union Stock Yards & Transit Company. Within this area there are no public streets, and very few City water mains. The City of Chicago cannot legally expend money for the laying of water mains within the confines of this private property.

During the past fifty years the City of Chicago has done everything within its legal power to provide proper fire protection for this area by bringing a tremendous supply of water up to its boundries; this water supply originating from a number of different sources. In fact,

the Bureau of Engineering has practically surrounded the Stock Yards with a veritable wall of water, as was demonstrated during the fire of May 19.

If the City of Chicago had not provided this wall of water around the boundaries of the Stock Yards, the fire department would have been helpless in combating the recent conflagration, and it would have reached far beyond the confines of the area with a resulting loss of life and property greatly beyond that which actually did occur.

THE FIRE

The fire of Saturday, May 19, 1934, according to reports, originated in the wooden cattle pens under the 43rd Street viaduct, about half way between Halsted Street and Racine Avenue, and probably caused by a lighted cigarette thrown from the 43rd Street viaduct and landing in some loose hay in the pens below. The fire was first discovered by a company watchman, and the first alarm was turned in at 4:21 p.m. The wind was blowing from the southwest, with an average velocity of 12 miles per hour, and maintained this general direction throughout the period of the fire. Although the general direction of the fire's travel was toward the northeast, it also spread south and west, due to falling embers igniting the dry wooden structures. Previous to the fire, a long period of hot, dry weather had prevailed, and as a result the wooden pens, overhead runways, and other frame structures, were in such a condition that they were easily and quickly ignited.

As stated before, the first alarm was turned in at 4:21 P.M. By 4:30 P.M., twenty companies with a pumping capacity of 15,250 gallons per minute, had responded. The first companies to arrive, engines 53 and 59, made their way into the cattle pens and connected to a hydrant on Texas Avenue, near 43rd Street. At this time, the fire had made considerable headway and was leaping over wide spaces and starting new fires. The fire was traveling with such rapidity that engines 53 and 59 were able to operate only five or ten minutes when their crews were compelled to abandon them. Both engines were destroyed. In attempting to drive cattle from the pens an employe of the Union Stock Yard and Transit Company lost his life near this location. Approximately 8000 head of cattle were destroyed.

In the area south of Exchange Avenue there were very few hydrants and no through streets. In the early stages of the fire, it was impos-

sible for the fire department to get close enough to the conflagration to do effective work, and within twenty-five minutes of its start the fire in the cattle pens, runways and sheds, south of Exchange Avenue, was out of control. By 4:45 P.M. the fire had reached north to the three-story brick South Exchange building, and the brick horse-barns along the west side of Dexter Park Avenue were on fire. At

FIG. 1. THE STOCK YARDS FIRE—LOOKING EAST

this time, the wind was driving the fire rapidly to the northeast, but at the same time, due to flying embers, it was also traveling westward.

Shortly after 4:30 P.M. the fire department attempted to make a stand on Exchange Avenue, west of Halsted Street, but was soon compelled to retreat from this position. After destroying the south Exchange building, the fire reached north into the eight-story brick

fire-proof Exchange building, on Exchange Avenue. This building is of fireproof construction, and although the interior was completely gutted, the building was not structurally damaged. The fire did not reach north of Exchange Avenue at this point.

On the southwest corner of Exchange and Texas Avenue was a frame structure known as sheep shed No. 2. The eastern portion of this structure took fire, but the fire department was able to prevent it reaching north across Exchange Avenue into what is probably the

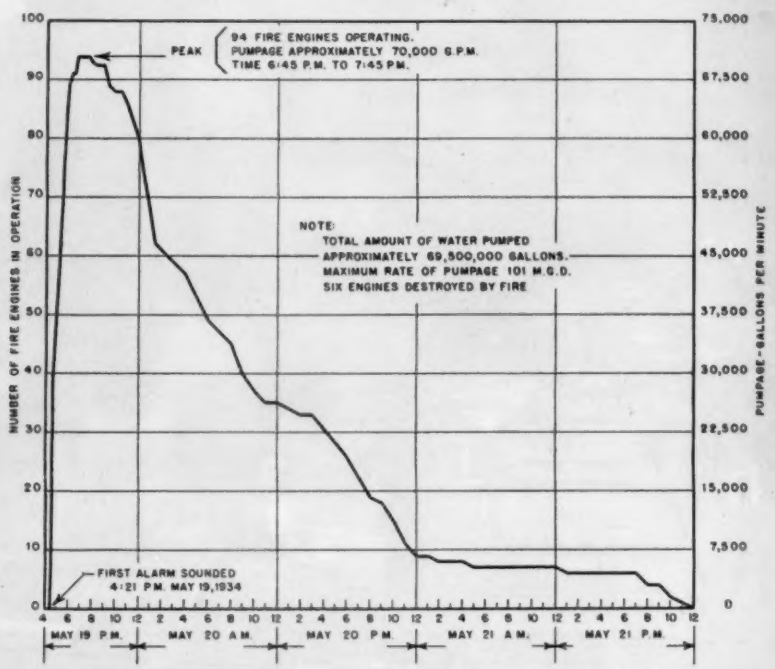


FIG. 2. CHICAGO STOCK YARDS FIRE, MAY 19, 1934. NUMBER OF FIRE ENGINES OPERATING AND PUMPAGE IN GALLONS PER MINUTE

most congested section of the Stock Yards. North of Exchange Avenue at this location is a thickly built up section consisting of rows of frame double-deck hog houses, extending without a break for nearly a half mile. It would have been impossible for the fire department to get into this area, and if these sheds had become involved the entire Stock Yards property north of Exchange Avenue would have been destroyed, and there is no telling how far the conflagration might have reached to the north.

The City had provided an 8-inch water main in Exchange Avenue, connected to hydrants at proper locations, and it is due to this that the fire department was able to block the northward travel of the fire at this point, and thus save property of tremendous value.

Shortly after this the fire department made a second stand in the vicinity of Dexter Park and Exchange Avenues, but they were soon driven from this position.

By 5 o'clock the fire had reached its greatest intensity, and was travelling rapidly to the northeast. Buildings on the west side of Dexter Park Avenue were burning rapidly, and the fire had reached to the north of Exchange Avenue, just west of Halsted Street. Engines No. 43 and 98 were destroyed on Dexter Park Avenue, and Engines 8 and 14 were destroyed on Exchange Avenue just west of Halsted Street. About this time the flames and embers had swept over Halsted Street, and involved the buildings on the east side of that thoroughfare. In this area the Dexter Park pavilion, the horse-barns, the buildings occupied by the Omaha Packing Company, the Drovers Journal, and the Mercury Manufacturing Company were destroyed, as well as numerous structures along the east side of Halsted Street. The Drovers National Bank Building, with the exception of the vaults, was gutted. The contents of the fire-proof Stock Yards Inn and the Breeders building were destroyed, and the buildings themselves considerably damaged. The fire-proof Live Stock National Bank building was somewhat damaged. The steel elevated railroad structure, in the vicinity of Exchange Avenue and Halsted Street, was greatly damaged for a considerable distance.

By 5 o'clock P.M. 52 fire engine companies with a combined pumping capacity of 41,250 gallons of water per minute, had responded. This is approximately twice the amount of water that had ever before been used at any fire.

By 5:30 P.M. the fire south of Exchange Avenue had reached westward to the rear of the Armour & Company office building on Racine Avenue. At this point it was stopped by the combined efforts of the City Fire Department, and the private fire brigades of Armour & Company. About this time a factory building, located at 3939 Union Avenue, about 900 feet northeast of the main fire, was ignited by flying embers. The fire in this isolated building was quickly under control and did not spread from this point.

By 5:30 P.M. there were 70 pumping engines on the ground and these engines had a pumping capacity of over 52,000 gallons per minute.

Shortly after 5 o'clock P.M., the fire department made its final stand on Halsted Street from 40th to 43rd Street and on the streets to the east of Halsted Street, and the fire was held in check along these lines. Shortly after 8 P.M., the wind velocity became less, and by 11 P.M. Saturday evening, the fire was under control. The fire was officially struck out at 11:47 P.M. on Sunday, May 20.

WATER SUPPLIED DURING FIRE

During the period between 6:45 P.M. and 7:45 P.M. on the afternoon of Saturday, May 19, there were 94 pumping engines operating in the fire area. These engines had a pumping capacity of approximately 70,000 gallons per minute, or at the rate of 101,000,000 gallons per day. This represents the greatest concentration of fire fighting apparatus ever known in history, and water was being delivered upon the burning area at a rate of probably three times that ever before recorded.

That the Chicago water supply system was able to deliver this tremendous and unprecedented amount of water at adequate pressures when called upon in this emergency speaks well for the farsighted planning and construction carried out by the Bureau of Engineering of the Department of Public Works in the years prior to this conflagration.

His Honor, Mayor Edward J. Kelly, who was on the ground throughout the period of the fire, requested all radio stations to make appeals to consumers to refrain from lawn sprinkling, and otherwise to conserve the water supply. These requests were broadcast shortly after the start of the fire and were repeated at short intervals. They were very effective, the people co-operating with the Mayor to such an extent that, although between 6 and 7 P.M., when the total pumpage for the City was at the rate of over one billion three hundred million gallons per day, pressure and supply were everywhere satisfactory. In fact, during the fire pressures all over the south side were increased.

The Water Pipe Extension Division had two crews of men early in the fire, who covered the entire Stock Yards District and inspected all city hydrants to see that they were in working condition. Every hydrant was found in first-class shape. This division also held in reserve at the Western Avenue Pumping Station two complete crews of men with trucks and equipment ready to close down valves or do any other work necessary in connection with broken mains. These crews remained in readiness until relased about 12:30 A.M. Sunday.

The Water Purification Division had a force of men covering the entire Stock Yards District during the period of the Fire. These men took samples of water from fifteen different points and analyzed it for residual chlorine. In every case where the water was taken from the City's water mains it was found that there was sufficient residual chlorine in the water to make it safe. In three instances where the water was taken from the private mains of the Union Stock Yards system little or no residual chlorine was found, and therefore additional samples were taken for bacteriological analysis. These men also noted the pressures on the pumping equipment of the Fire Department, and reported that in every instance the pressure was satisfactory. These men remained on duty throughout the night.

Mr. Cullen, Mr. Garrity, Mr. Eddy, Mr. Anglim and their assistants, of the Water Pipe Extension Division; Mr. Gorman, Mr. Gerstein and their assistants, of the Division of Water Purification, and many others of the Bureau of Engineering were on the ground ready to do their part throughout the period of the fire.

Colonel Henry A. Allen and Mr. Frank McDonough, the heads of the Operating Division, and their assistants, deserve credit for the way the pumping stations operated in this emergency. All the various equipment in the stations was in operating condition and delivered water up to designed capacity. The operating forces in the various city pumping stations met every requirement.[1]

(*Presented before the Illinois Section meeting, April 11, 1935.*)

[1] In preparing the foregoing paper free use has been made of the valuable reports published by the National Board of Fire Underwriters, the records of the Bureau of Engineering of the City of Chicago, and the records of the Chicago Fire Department.—L. D. G.

AN ANALYTICAL METHOD OF DETERMINATION OF LEAKAGE IN WOOD STAVE PIPE

By A. A. Surin

(Leningrad Scientific Research Institute of Water Supply, Sewerage, Hydrotechnical Constructions and Engineering Hydrogeology, Leningrad, U. S. S. R.)

It is of essential importance to know precisely the extent of wood stave pipe leakage in order to estimate the amount of water wasted along a pipe line, since waste of water must be seriously taken into consideration in pumping operations.

In America, where wood stave pipes were first constructed, and where they have become a product of mass manufacture, the specific daily leakage e is generally considered per mile of length of pipe L and per inch of its diameter D. The total daily leakage along any pipe line may be represented as follows and designated as formula 1

$$Q = eDLt \qquad 1$$

in which t is the number of days during which leakage occurs.

In the above formula the internal pressure of water within the pipe is ignored as though the leakage from a pipe was independent of the pressure of the water contained therein. This is contrary to fact. Actually, this formula is adequate for computing working pressures, or pressures approximating the latter. But the absence in formula 1 of the function of internal pressure p in pipes has prompted some engineers, of other nationalities, to introduce this internal pressure in formula 1 as a multiplier. This modification gives formula 2.

$$Q = eDpLt \qquad 2$$

In our opinion, formula 1 is better suited for application to acceptance tests of wood-pipes than formula 2. According to formula 1 all wood-pipes must be constructed so as to give an approximately equal specific leakage e when put to use at the calculated pressure. Hence it would be a mistake, for instance, to construct a pipe to operate under a pressure of 6 atmospheres (88.2-feet) allowing for a

leakage double to that which takes place in a pipe with an internal pressure of 3 atmospheres (44.1-feet). The question of internal pressure may arise when pipes constructed for a certain definite internal pressure are actually put to use under another pressure. Will the magnitude of leakage, in that case, be proportionate to the first power of pressure, as stated in formula 2? It is this question which the present article attempts to solve.

Pipe leakage may develop from three causes.

(1) Leakage through chinks (cracks), formed by a slackening of the longitudinal joints and transverse junctions owing to the sagging or deflection of pipe staves under the action of the internal pressure of water within them.

(2) Leakage through pores in the staves and capillary chinks between imperfectly planed and poorly-adjusted staves in the joints and junctions, and

(3) Leakage owing to flaws in the staves (unsatisfactorily mortised knots, damaged staves, etc.), to loosened joints and junctions (unsatisfactory assemblage of the pipes).

Let us now analyse the action of each of the above mentioned causes on the structure of the formula, emphasizing the dependence of the leakage upon the internal water pressure.

(1) The first cause—the sagging of the staves and the opening of the joints and junctions under the action of internal pressure depends upon the magnitude of the internal water pressure in the pipe. If we look upon a stave as a continuous beam, uniformly loaded and the spiral binding wires acting as supports—the sagging of this stave will be proportional for any intermediate span, as well as for the cantilever end parts of the stave to the first power of the actual pressure p_0. Thus the maximum sag f of the intermediate span of the staves may be expressed by the formula:

$$f = Ap$$

in which A is a constant coefficient for the given pipe.[1]

The magnitude of sag f in the staves, figure 1, increases by

$$\delta = \pi (D + 2f) - \pi D = 2\pi f = 2\pi Ap \qquad 3$$

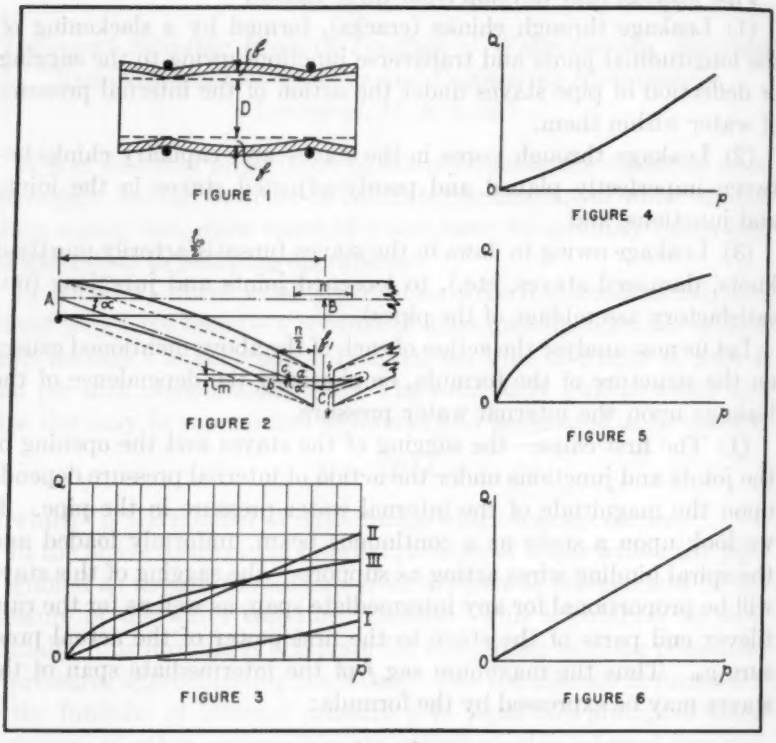
the perimeter of the cross-section of a pipe with a diameter D at the middle point between adjoining bands.

[1] For more details see N. Raborsky, Holzdaubenrohre, pp. 17–20, 1926.

If we call l the distance between two adjoining bands, the area of the joints opening in a span between adjoining bands will be:

$$w_1 = \frac{\delta \cdot l}{2} = \pi A l p \qquad 4$$

Formula 4 might be used for determining the flow area in the pipe junctions, if the wood did not swell when soaked. Soaking can



FIGS. 1–6

either seal the joints completely or partially reduce the area of chinks and crevices. But, in any case, if the staves are disjointed, due to the sagging, the area of chinks and crevices will be proportional to the actual pressure p_0. We may assume, for rough evaluation, that the actual pressure p_0 is equally distributed for a pipe laid above the ground, to the internal manometric pressure in pipe p. For a pipe placed underground the actual pressure will be equal to the difference

between the pressure p_0 and the external pressure of the ground on the pipe p_1, i.e. $p_0 - p_1$.

In the cross-joint or butting of the staves, figure 2, one usually inserts a tongue that helps to keep the joint water-proof so that the sag in the staves does not expose the joints, but a certain shifting of the tongue takes place in it, and consequently a corresponding slackening of the seams which renders possible the oozing out of water. According to figure 2 the joint plate n will tend to compress the upper fibres in the butt, that is, the fibres placed above the plate, to mash the lower fibres in the stave reducing them to the value m at the extreme points, that is at the butt of the stave and the end of the plate. The compression of the fibres will certainly not attain the value m because the jointing tongue will bend and move somewhat outward from its mortises (sockets), but we may assume that the water passage which was formed is proportional to m; and the value of m, by analogy in the triangles $A\ B\ C$ and $a\ b\ c$ will in its turn be proportional to the sag f_1; and the latter will be proportional to the actual pressure p_0. In other words, it seems quite reasonable to assume, that in the case of a butt joint in the staves the value of the opening, for the passage of water, in the cross-section of the joint is proportional to the actual pressure p_0; it may, therefore, be expressed by the following formula:

$$w_2 = Bp_0 \qquad 5$$

As appears from calculations the area of the opening of the seams and joints resulting from a bend in the stave, created by an internal pressure of water in the pipe, is extremely insignificant, even when the swelling of the wood is not taken into consideration, and is expressed in small fractions of a millimeter. It follows from this that the leakage of water through the chinks, developed at the bend, proceeds in obedience to the law governing the motion of water in channels of the capillary type, and is expressed by the formula

$$v = k_1 i = k_1 \frac{p}{C} = kp \qquad 6$$

in which k_1 is the coefficient of velocity,

i = the hydraulic gradient equal to the quotient of the stave C and k

k is a coefficient equal to k_1/C.

Pressure p in formula 6 expresses in all cases the manometric

pressure inside the pipe. This proposition is perfectly clear for a pipe laid above the surface of the ground and remains equally clear for pipes placed in the ground, since the air in the pores of the ground has a pressure about equivalent to that of the external atmosphere, and the pores themselves allow a sufficiently free passage to the small quantity of water leakage from the pipe. Even when a pipe is submerged in water to its upper generatrix, the pressure of outside water is so insignificant as compared to the internal pressure of water in the pipe, that it may be omitted from further consideration.

The amount of water leakage from the pipe is equal to the flow area multiplied by the velocity, hence according to formulae 4, 5 and 6.

$$Q = (xw_1 + yw_2)v = (x\pi Alp_0 + yBp_0)kp = \alpha \cdot p_0 \cdot p \qquad 7$$

in which α is the constant coefficient of the given pipe;

x is the spanl between the bands in the pipe

y is the number of cross-joints in the same section.

When we have to deal with a pipe laid above the surface of the ground $p_0 = p$ and formula 7 will become

$$Q_1 = \alpha p^2 \qquad 8$$

Since the internal pressure of water p is always considerably greater than the external pressure p_1 of the ground on pipe, formula 8 also gives the approximate quantity of leakage of water due to the above mentioned first cause. For under ground pipes it is, therefore, permissible to assume that the water leakage from wood pipes, through seams and joints, created by the deflection of staves under the action of the internal pressure of water is proportional to the second power of the internal manometrical pressure p of the water within the pipe.

(2) The second cause of water leakage from wood-pipes, attributed to the existence of pores in the pipes staves and the imperfect planing of adjacent staves in seams, depends slightly, or not at all, on the internal water pressure in the pipe if we designate the total area of percolations in the walls of the pipe from the above mentioned cause by w_3, we may express the value of the leakage by means of the following formula:

$$Q_2 = w_3 \cdot v$$

In this formula v is expressed by formula 6, since the pores in the

staves and the looseness of the joints, in the seams, form channels of a capillar type through which the water leaks; therefore

$$Q_2 = w_3 \cdot k \cdot p = \beta \cdot p \qquad 9$$

in which β is the constant coefficient of a particular pipe.

In other words the water leakage created by the second cause is proportional to the first power of the manometrical pressure p of the water in pipe.

(3) The third cause of the water leakage from wood-pipes is the defectiveness of the staves, such as the presence of holes where knots have dropped out, the effects of some injury caused to staves or noticeably loose seams in the junctions of the pipes. All such defects favor the passage of water, by forming orifices of considerable cross section. The velocity of water flowing through these is expressed by the formula

$$v = k_2 \sqrt{i} = k_2 \sqrt{p/C} = \frac{k_2}{\sqrt{c}} \cdot \sqrt{p} = k_3 \sqrt{p} \qquad 10$$

in which k_3 is a constant coefficient.

The total area of the orifices w_4 is independent of the internal pressure p. Therefore the water leakage created by the third cause may be expressed by the formula

$$Q_3 = w_4 \cdot k_3 \cdot \sqrt{p} = \gamma \sqrt{p} \qquad 11$$

in which γ is a constant coefficient for the given pipe.

The total leakage from pipes due to all three causes as represented by formulae 8, 9 and 11 may be summarized as follows:

$$Q = \alpha p^2 + \beta p + \gamma \sqrt{p} \qquad 12$$

If we give a graphical representation in coördinates of the individual members of formula 12, in which the abscissae represent the internal pressure in pipe p, and the ordinates the value Q of the water leakage from the pipe (see figure 3); the first member containing p^2 will be expressed by the parabola 0–I, with up-turned concavity; if we express this parabola by the equation generally accepted in analytical geometry

$$y^2 = 2 s x \qquad 13$$

we have in the given case

$$Q_1 = \alpha \cdot p^2$$

or

$$p^2 = \tfrac{1}{a} \cdot Q_1 \qquad 14$$

that is the axis of pressure p corresponds to the y axis and the axis of leakage Q corresponds to the x axis.

The second member of the formula 12 will be represented by the straight line 0–II by means of the equation 9, and its third member by the curve 0–III, essentially differing from the curve 0–I owing to its convexity being turned upwards; it is expressed by means of equation 11.

Experiments with pipes give the total water leakage as

$$Q = Q_1 + Q_2 + Q_3$$

and in conformance with their results one can readily express, by the coördinates p and Q, the curve of total leakage water without subdividing it into the three above mentioned categories. The aspect of this curve will depend upon the character of the prevailing leakages. For instance, if the results of the experiments are represented by the curve on figure 4, we shall have to assume the presence of considerable leakage through chinks caused by the opening of the seams and joints during a deflection of the staves under the action of the internal pressure of water. Such a curve shows that the staves of the pipe have slackened either owing to their thinness, the insufficient resistance to deflection by the material itself or because of the interval between the bands being too large. According to preliminary computations such an aspect of the experimental curve is hardly probable, since in a normally constructed pipe, made of good material, leakage through seams and joints owing to this cause is inconsiderable and cannot exercise any appreciable action on the aspect of the total curve. If the experimental curve takes on the aspect shown on figure 5 one must assume the existence of considerable leakage through flaws in the staves or loosened seams and joints. The medium between the experimental curves we have analysed would approach a straight line. The total leakage represented by the straight line on figure 6 can only occur in two cases:

(1) When the overwhelming amount of wasted water can be explained by its flowing through the pores of the pipe staves and through the capillar chinks between the roughly planed and loosely joined staves.

(2) When in face of the existence or absence of the aforementioned

leakage we are confronted with two other varieties of leakages: Q_1 and Q_3 standing in such relationship towards each other that, owing to a reciprocal opposition of the flexures of the curves representing them, we finally obtain approximately a straight line. According to preliminary computations this can happen only in extremely rare cases.

All the above mentioned formulae are deduced under the assumption that the pipe is submitted from all sides to an equal uniform pressure and that both the water and pipe are without weight. In reality these assumptions are contradicted by the following facts:

1. The ground exerts pressure on the pipe with varying degree from different sides.
2. Water possesses weight.
3. The pipe itself also possesses weight.

Many works have been directed to the examination of these factors. Unquestionably the above indicated factors are partially responsible for the deformation of the pipe; they tend to convert it into some kind of ellipse with a big horizontal axis. Likewise it cannot be disputed that in a correctly built and adequately laid pipe these deformations are so slight that the leakage is confined to the general aspect herewith analysed and that the question under discussion cannot be solved more accurately.

According to preliminary calculations and the analysis which has been performed, the first member in formula 12 may be left out owing to its insignificance. This simplifies formula 12, reducing it to:

$$Q = \beta \cdot p + \gamma \sqrt{p} \qquad 15$$

In adequately constructed and carefully laid pipes the first member of formula 15 will prevail on account of its numeral value; whereas for an inadequately constructed and poorly laid pipe—it is the second member of this same formula which prevails. Formula 15 enables us to determine water leakage from a pipe under any pressure whatever if the leakage was previously measured under some two other pressures. For instance if some like of the pipes tested for leakage gave, under a 6 atmosphere pressure, a leakage of 1280 gallons per hour, and under a 5 atmosphere pressure—1150 gallons per hour, we may put down, according to formula 15

$$1280 = 6 \cdot \beta. + \sqrt{6 \cdot \gamma}$$

and

$$1150 = 5 \cdot \beta + \sqrt{5 \cdot \gamma}$$

By simultaneously solving both equations we have $\beta = 50$ and $\gamma = 400$, thus giving to formula 12, for the given pipe, the following aspect:

$$Q = 50\ p \times 400\ \sqrt{p} \text{ gallons per hour}$$

Under a different pressure in the pipe, say 3 atmospheres the leakage will be

$$Q = 50 \times 3 + 400\ \sqrt{3} = 150 + 693 = 843 \text{ gallons}$$

As appears from calculations, the main leakage from the pipe under examination depends on a third cause, that is: on defects in the staves and on unsatisfactory assembling. When the pressure is changed from 6 atmospheres to 3, the leakage is only reduced $1280 \div 843 = 1.5$ times.

Formula 15, as appears from figures 3 to 6 and as deduced from the example gives us an initial measure of the diverse kinds of water leakages from wood pipes and establishes the nature of the defects of the latter. In this lies its intrinsic merit.

Engineer A. J. Popkov, based on research performed by him in the Leningrad Scientific-Research Institute of Water Supply, Sewerage, Hydrotechnical Constructions and Engineering Hydrogeology, gives in his work entitled "The application of wood-pipes" an empirical formula for determining specific water leakage using pressure as an index.

$$e = Ep^n \qquad 16$$

He comes to the conclusion that in the case of five wood pipe systems laid in the U. S. S. R. and investigated by him the coefficient E has a constant value for each given pipe, but that it has a different value (of a sixfold variation) for different pipes, whereas the power index n, for any diameter, has a constant value of approximately 2/3 for all the pipe lines under consideration.

The total water leakage from all of the pipe line, assumes in reference to pressure and with the application of formulae 1 to 16, — the following aspect:

$$Q = Ep^n\ DLt = \Sigma p^n \qquad 17$$

in which Σ is the constant coefficient of the given pipe line.

By comparing formulae 15 and 17 we see that formula 15 consists of two members, one of which has a first power of pressure p and the other the same pressure with power reduced by half; it is obvious that owing to the limited amplitude of pressure fluctuations in wood pipes, one may fit into this formula the approximate monomial formula 17 having for the given pipe a constant coefficient Σ and a power index n between 1.0 and 0.5; in addition to this the power index n must, on the strength of the aforestated considerations, come the nearer to 1.0 the more adequately the pipe line is constructed and assembled. If the pipe line is inadequately constructed and assembled n approaches 0.5. With pipes of a more or less uniform quality the value n may be less variable. If we consider the above mentioned example in its application to formula 17 we have two equations,

$$1280 = \Sigma\, 6^n$$

and

$$1150 = \Sigma\, 5^n$$

A simultaneous solution of both equations gives the following values: $\Sigma = 447$ and $n = 0.587$. That is; formula 17 for the instance under consideration becomes

$$Q = 447\, p^{0.587}$$

Therefore in the case of work with wood pipes under a pressure of three manometrical atmospheres the value of the leakage shall be $Q = 447 \times 3^{0.587} = 849$ gallons per hour, which approximates the value computed by means of formula 15.

Formula 17 with a certain power index n achieves simplicity, but it gives no indications as to the nature of the defects of the pipe, whereas formula 15 does. But if we take into consideration that as a rule the coefficients Σ and n may be different in it for different pipes, the simplicity of formula 17 loses its desirability, since the application of both formulae demands, as appears from the example we have considered, two preliminary leakage test observations of wood pipes under different water pressures.

NEW FILTRATION PLANT AT ALBANY, NEW YORK

By Ezra B. Whitman

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Previous to the construction of the new water supply for the City of Albany, the water supply had been obtained from the Hudson River. After filtration the water was pumped through low lift and high lift pumps. The low lift pumps discharged into Bleecker Reservoir at an elevation of 242 feet. The high lift pumps discharged into Prospect Reservoir at an elevation of 297 feet. The water from this higher reservoir and in the high level system did not furnish sufficient pressure to adequately serve the high level districts in the western part of the City, pressure being barely sufficient to deliver water to the second floor at the highest point.

The new system, bringing water a distance of 20 miles from the Helderberg Mountains southwest of the City, was designed to deliver water by gravity into a new distribution reservoir at an elevation of 351 feet. The new system has therefore dispensed with all pumping and it will save the citizens of Albany, through operating and other costs, between three hundred and four hundred thousand dollars per year, with a water consumption of 20 million gallons per day. When this consumption is increased to 30 million gallons per day, the annual saving would be well over one-half million dollars per year.

TYPE AND SIZE OF FILTER PLANT

The type of plant selected for Albany is the rapid sand or mechanical type and the present construction provides for an average rate of filtration of 32 million gallons per day, and the plant can be operated for short periods at an increased rate of 40 million gallons per day. The plant is so designed as to provide for an extension of all units to an ultimate and normal rate of filtration of 64 million gallons a day, which can be increased for short periods to a rate of 80 million gallons per day.

Certain features of this plant are constructed at the present to the size required for the ultimate capacity of the completed plant. These

features include influent and effluent mains, all water channels, clear well and wash water tanks, chemical storage, chemical handling and dosing equipment and laboratory equipment. The features which require expansion when the capacity of the plant is enlarged, include aerators, mixing basins, coagulating basins and filters.

LOCATION

The Alcove Reservoir, from which the water is drawn to feed the filtration plant, has an overflow elevation of 618 feet A.M.T. The lowest point at which water can be drawn down to deliver water to the filter plant, we figured as 572 feet A.M.T. Water can be drawn down even lower than this by by-passing the aerators; however, the greater part of the available storage in the Alcove Reservoir will be above 572 feet elevation.

The elevation at which water will be delivered to the City of Albany will be at 350 feet, which will furnish practically the entire City area with water by gravity. The plant could have been built at the Alcove dam, but such a location had numerous disadvantages. The great distance of this dam from Albany and from the nearest railroad would have created difficulties particularly during the winter season in transporting the operating forces to and from the plant, and would have caused increased cost and difficulties in the delivery of chemicals. The necessity, also, of placing the plant below the minimum draft line of the Alcove Reservoir, would have reduced the delivery capacity of the supply conduit during those periods when the water level in the Alcove Reservoir is at or near the spillway level.

The problem in locating the filter plant was to find a point as close to the City as possible where the elevation of the ground would be approximately the same as the hydraulic gradient from Alcove Dam to the City. The new supply is brought in from the south and if high ground could have been found on the southern side of the City close to it, that would have been the logical place to locate the filter plant. The only high ground, however, that was available was on the other side of the City, so that it would have been necessary to have carried a raw water line entirely through the City, with a filtered water line of equal capacity feeding back into the City.

A storage reservoir of 100 million gallons capacity was located on this high ground to the north of the City. The filter plant, however, we found necessary to locate ten miles south of the City, as this was the only point where ground of sufficient elevation could be found

upon which the filter plant could be located. This location was near the town of Feura Bush where a railroad is located one half mile from the plant and where supplies for the filter plant can be brought in by rail.

This location was on a rock plateau, where almost the entire excavation for the plant was made in solid rock. In many places the rock was on the surface of the ground and the deepest covering of earth was not over two feet, and, in many cases, was just a few inches in depth. This rock excavation made the construction work considerably more expensive than is usually encountered in the construction of a filter plant. The deep pipe lines for the drains and the effluent pipe were in trenches 18 to 20 feet or more in depth and the plant itself had rock excavation as deep as 16 feet for some parts of the plant.

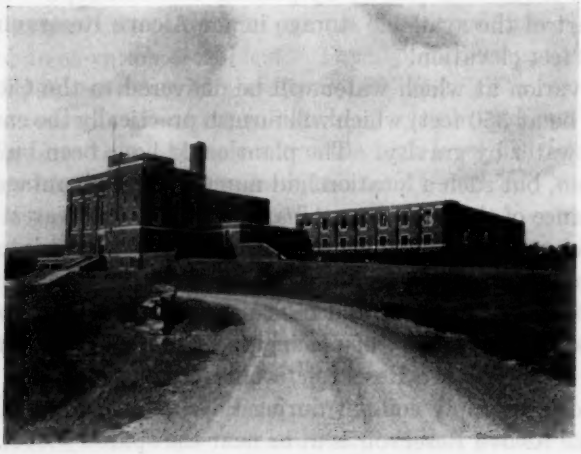

FIG. 1. COMPLETED FILTER BUILDING

CONTROL OF WATER ENTERING PLANT

Water drawn from the Alcove Reservoir enters the plant from the South through a 48-inch iron conduit. At the southern limits of the plant is a valve vault containing connection of the present 48-inch conduit with the 60-inch influent main supplying the plant. An additional connection to this 60-inch main is provided for the future connection of a second 48-inch supply line when additional water is brought in from Catskill Creek or Ten Mile Creek. Within this vault

there is also a blow-off valve to permit the emptying of the line and a pressure relief valve to prevent the 60-inch influent main from being subjected to excessive pressure due to possible water hammer. The 60-inch influent main continues from the valve vault through the basement of the Chemical Building at which point there is installed an influent Venturi meter 60 inches by 30 inches in size to record the amount of water entering the plant. The outlet cone of the Venturi meter contains a Dow disc butterfly valve actuated by a water level controller which throttles the water passing through this meter to meet the requirements of filtration so as to maintain a fixed water level over the filters regardless of the quantity of water passing

FIG. 2. AERATOR BASIN DURING CONSTRUCTION

through the filters. Where the 60-inch influent line passes through the Chemical Building, provision is also made for the application of either lime, alum or sulphuric acid to the raw water. The 60-inch line terminates at the aerator building.

AERATORS

The 60-inch main delivery line leads to the aerator chamber and forms a header along one end of this aerator chamber. Five lines of pipe are laid from this header into the aerator basin. These lines are 36 inches in diameter, tapering to 24 inches, and are of sufficient size to be extended to provide an aerator system of double the present

capacity. At the entrance to the aerators, a by-pass connection is provided which permits at any time the elimination of the aerators from the filtration process.

Owing to the difficulties which have been experienced at the old Albany filtration plant with the open air aerators and the formation of slush ice, it was determined to cover the entire aerator basin to prevent these difficulties. The nozzles are tapped into the top of the five lines which feed the aerator nozzles and a study of various aerator plants which have been installed throughout the country, showed that the space for aeration varied from 167 to 675 square feet per million gallons.

The aerator basins for the Kensico and Ashokan supplies for the City of New York were each designed with 193 square feet per million gallons in the aerator basins. We used in our design 200 square feet and the nozzles were placed $20\frac{3}{8}$ inches apart on the feeder lines and each nozzle has approximately 40 square feet for its operation. The nozzles themselves were $1\frac{5}{8}$ inch in diameter and are capable of discharging 130 gallons per minute with 11 feet head at the nozzles. The interiors of the nozzles have four vanes which give the water a whirling motion so that when it leaves the nozzles it is broken up into a fine spray, thus bringing about excellent aeration.

MIXING BASINS

Water is led from the aerating basin through a rectangular reinforced channel and conduit to the mixing basin. These mixing basins are three in number and are so arranged that they can be operated either in series or in parallel. With the three basins operating in series, the loss of head from the inlet to the outlet was figured as 2.82 feet. Actual experience with these basins checks very closely the original computations for the loss of head. These basins will hold a 20 minutes supply of water at a rate of 32 million gallons per day. When the water first enters each basin, it passes a series of up and down baffles through the first channel which thoroughly mixes the coagulant with the water. After passing through this first channel, it then travels horizontally back and forth across the basins arranged with around-the-end baffles until it reaches the point of outlet.

If the tree basins are operated in series, the total length of travel through the mixing basins is 1200 feet. The velocity of flow through the basins at the rated capacity of the filter plant is 0.9 foot per

second. The capacity of these basins was based largely on the result of experiments made in Baltimore where it was found that the resulting effluent, after mixing for 5 minutes and holding water 7 to 8 hours in the coagulating basins, was equivalent to the effluent obtained by 30 minutes mixing and holding the water in the coagulating basins for only 2 hours. Thorough mixing makes it possible to cut down the capacity and therefore the cost of the coagulating basins, the cost of increasing the capacity of the mixing basins being much less than would be the cost of increasing the size of the coagulating basins.

The baffles in the mixing basins are made of wood and are so constructed that the velocity of flow through them may be varied by widening or narrowing the channels between the baffle walls. If the rate of flow through a mixing basin is too rapid, the agitation prevents the formation of floc while, if the rate of flow is too slow, it tends to precipitate the floc in the mixing basin itself.

Experiments made at Reading, Pennsylvania, showed that the best results could be obtained in a mixing basin where the velocity of flow was kept above 0.25 and below 0.9 of a foot per second. We designed our mixing basins therefore so that the rate of flow would be a maximum of 0.9 foot when the filter plant was operating at its full capacity. At partial capacity, this flow drops below this rate, but at all times it will stay within the limits found to be desirable in the Reading experiments.

COAGULATING BASINS

The water is led from the mixing basin by means of reinforced concrete conduits to the far end of the coagulating basins and enters the coagulating basins through three hand operated sluice gates. The basins themselves were designed to have a detention period of 2.5 hours at the maximum rate of flow. They are constructed on the straight flow principle, the water entering at one end and leaving at the other.

At the inlet end, the basins are approximately 18 feet deep, while at the outlet end, they are 10 feet deep, the water being drawn off at the surface over a submerged weir. At the inlet end of the basins, about four feet from the entrance sluice gates, a brick baffle wall is constructed. Directly opposite the inlet gates, this brick baffle wall is solid. Throughout the rest of the wall, there are perforations in the bricks and these perforations tend to distribute the flow of

water throughout the entire cross section of the basin, so that there are no areas of dead water. These perforations are designed so that the sum of the openings is equal to 2½ per cent of the area of the basin and the water passes through the perforations with a velocity of approximately one foot per second.

In cleaning these basins, there is a drain at the inlet end and the sloping bottom from the outlet end to the inlet end, making it a very simple matter to wash the water down this slope to the outlet drain. Actual experience of the filter operators shows that when these basins are approximately half filled with sediment, they can be cleaned in twenty minutes or less and be ready for filling and operation at the end of this time.

Experience has shown that in climates as cold as that of Albany, it is desirable to cover the coagulating basins so as to prevent formation of ice which tends to interfere with the operation of the basin and makes it practically impossible to clean a basin on which there has been a formation of heavy ice. In the warmer climates in the more southern latitudes, it is not necessary to cover the basins. There is a reinforced concrete cover with two feet of earth over the concrete. The interior of the basins is provided with electric lights and a system of piping and hose connections is installed for the purpose of cleaning and flushing the basins. Additional coagulating basins can be constructed to the west of the present units to provide for the enlargement of the plant; and, the existing supply, and effluent channels can be extended.

FILTERS

After leaving the coagulating basins, the water enters a reinforced concrete channel which has been built entirely around the filters themselves. This arrangement makes it possible to have the influent supply conduits kept out of the filter gallery, keeping the gallery much more clear and open than it would be otherwise. The filter boxes themselves are 46.5 by 30.5 feet in area and have a capacity of 4 million gallons per day at the rate of filtration of 125 million gallons per day. The filter boxes themselves are 12¼ feet in depth and have 18 inches of gravel graded from 2½ inches over the strainers to a gravel passing 10 mesh to the inch at the top. On this finer gravel are placed 30 inches of sand with an effective size of 0.33 mm. and a uniformity coefficient of 1.75.

The filter bottoms are concrete channels draining to a large center

drain. These channels are covered with reinforced concrete and brass piping extends through this covering from the channel to the

FIG. 3. FILTER GALLERY DURING CONSTRUCTION

top of this floor and a cast brass strainer is screwed on the top of these brass tubes. There are 32,500 of these strainers in the eight

filters. The wash water troughs, fourteen in each filter, are placed at an elevation of 27 inches above the sand surface and the clear distance between the troughs is 4.57 feet. The rate at which the wash water is applied is with a vertical rise of 24 inches per minute. The effluent from the filter units discharges into two 48-inch cast iron mains, one on each side of the filter gallery, and these two mains converge into a single 60-inch effluent main which passes through the basement of the chemical building to discharge into the two million gallon clear well. A 60-inch by 30-inch Venturi meter is installed on the main to record the amount of filtered water delivered by the plant to the City. The filter units are so arranged that additional units can be constructed by the extension of the filter building to the west and by extending the existing inlet channel and effluent and wash water piping.

WASH WATER SYSTEM

Wash water can be provided either through means of a large wash water pump or from a wash water tank. Had it been decided to use a wash water pump, it would have required a pump of 30 million gallons per day capacity and a 400 H.P. motor. This would have greatly increased the demand element for the electricity used at the plant and the pumps themselves would have been operated not more than four or five times a day under ordinary circumstances and then for only a period of five minutes or less.

By building duplicate elevated tanks over the mixing basins of 252,000 gallons capacity, we could install a three million gallon pump with a 40 H.P. motor and keep this pump in operation for a large part of the day, keeping the tank filled. These pumps are automatically operated, starting when the water level in the tanks reaches a predetermined level, and shutting off when the tanks are filled. Another advantage of having a tank instead of a pump is that if the electricity should be cut off for any reason, it would still be possible to operate the filter plant and wash three or four filters before the plant would be out of operation.

The flow required for washing is at the rate of 21,000 gallons per minute and the tank bottom is 26½ feet above the sand surface. The rate of flow from the wash water tank to the filters is controlled by a rate controller designed to pass normally 30 million gallons of water per day, but capable of adjustment by 10 million gallons daily in either direction. There is also a Venturi meter on this pipe line from

the wash water tanks which makes it possible to measure accurately the amount of wash water which is being used at the plant.

THE FILTERED WATER RESERVOIR

After leaving the filters, the water enters two 48-inch cast iron effluent mains and is led into the filtered water basin which is built away from the filter building and has a capacity of two million gallons. This basin is built in two sections and in order to raise the pH of the filtered water, lime is added to the water in the effluent piping before it enters the clear water basin. These clear water basins become sedimentation tanks for settling out the lime before it enters the conduit leading to the City. One of the clear water basins can be kept in operation while the other is being cleaned. This clear water basin not only acts as a sedimentation basin after the application of lime, but it can be drawn on to supply several hours flow to the City in case it should be necessary to shut down the plant.

PLANT WATER SUPPLIES

The wash water system has already been described. In addition to the wash water system, there are two other systems, one for flushing water and the other for domestic supply. The flushing water system was installed for the purpose of cleaning the coagulating basins, filters, aërator basins, and for supplying water to fire hydrants around the plant. This system uses unfiltered water supplied by an electrically driven centrifugal pump, with a capacity of 750 gallons per minute against a head of 173 feet.

On this system are provided the three way hose connections in each of the coagulating basins and 2 three way hose connections for the aërator. A single hose connection is provided for each of the filter units. The domestic system furnishes filtered water throughout the plant. Two Typhoon pumps of the duplex gearless chain driven type, pump the water from the effluent main. Each pump has a capacity of 2,500 gallons per hour. They deliver the water into a pneumatic pressure tank 6 feet in diameter and 24 feet long, containing a total volume of 5,000 gallons. The pumps are automatically started and stopped by pressure switches which maintain pressures of from 75 to 100 pounds on the system.

Water from this system is used for the operation of all hydraulic valves, the operation of the dry feed chemical machines and to supply water for use in the garage. Water from this system, reduced to a

pressure of 50 pounds, is also supplied for the chlorine machines, the two heating boilers, hot water heater and for use in the laboratory, for the plumbing fixtures and for drinking fountains located in the various buildings.

HEATING SYSTEM

All of the buildings are heated by steam supplied from two 150 H.P. Heggie Simplex boilers. These boilers are provided with oil burners and heavy fuel oil is stored in two steel tanks each of 8,000 gallons capacity located outside the buildings. The smaller rooms are heated by direct radiation. The filter room, the filter pipe gallery, the lime storage room and the aerator buildings are heated by large unit heaters. The garage, pump room, aerator control room, mixing basin room, wash water tank room, chemical machine room, chlorine storage room and the laboratory are heated by small unit heaters.

SUPERSTRUCTURE

As previously described the clear well and coagulation basins are covered structures which are completely under ground. The remaining units of the filter plant are enclosed with building superstructures which form a pleasing building group. The sub-basement of this building contains the main influent and effluent pipes with minor pipe connections. The basement of this building contains auxiliary pumping equipment, acid application equipment, boiler room, machine shop and storage room. The first floor of the chemical building includes the superintendent's office, chemical feed machines, chlorine machines and chlorine storage space. The second floor of the chemical building provides space for the storage of lime and for the distribution of alum which is stored in hoppers located on the third floor. A men's toilet and locker room is provided on a mezzanine between the second and third floors.

Immediately behind the chemical building, is the wash water building, the lower portion of which houses the mixing basins. The second floor of the wash water building contains the large concrete wash water tanks which are supplied with filtered water by duplicate wash water pumps located in the basement of the chemical building.

Immediately behind the wash water building is the large filter building which is one story in height, and to the north of the filter building is the aerator building, which is also one story in height.

The low temperatures prevailing in winter made it necessary to completely enclose the aerator basin with this building. In front of the aerator building is the laboratory building, the second floor of which is used for laboratory purposes. The first floor of this building contains the controllers for the operation of the aerator laterals.

CHEMICAL STORAGE AND APPARATUS

Provision is made for the use of alum, lime, sulphuric acid and chlorine in the filtration process. The alum is stored in twelve concrete bins on the top floor of the chemical building. Each bin is of hopper shape, being 9 feet square at the top, 6 inches square at the bottom, and 10 feet 3 inches deep. The twelve bins have sufficient capacity to store 207.6 tons of rice alum. This storage is equivalent to approximately twenty-eight days' supply of chemical when applied at a maximum dosage to 40 million gallons of water per day. A bucket elevator is installed to lift the alum from the roadway level up to the level of the bins and from the top of this elevator is a screw conveyor which transports the alum to any bin which requires filling. The bucket elevator has a capacity of 12 tons per hour and the screw conveyor a capacity of 15 tons per hour. On the second floor of the chemical building, is a weigh larry with beam scale and hopper which provides for the weighing and distribution of alum from the concrete bins to the small hoppers mounted on the chemical machines which are on the first floor of this building. The weigh larry has a capacity of five hundred pounds. Three dry feed chemical machines of the Wallace and Tiernan Company manufacture are installed for the application of alum. These machines are of the dry disc feed type and on each machine is mounted a hopper of sufficient capacity to supply alum for a period of about eight hours when fed at a maximum rate of dosage at a filtering rate of 40 million gallons per day. The dry chemical from each machine is deposited into a small hopper which is irrigated with a stream of water and the solution is conducted by means of an eductor by a system of piping which permits the chemical to be applied either into the 60-inch influent main or into the entrance of the mixing basins.

Provision is made for the storage of lime on the second floor of the chemical building. This floor is designed to carry a load of 225 pounds per square foot or approximately a four foot height of bagged lime and the available floor space permits the storage of about 74 tons of this material. The lime is delivered in bags at the roadway

TABLE 1

Final estimate, Albany Filter Plant contracts

Contract No. 8. Filter Plant Substructure and Superstructure: Opening of bids, November 11, 1929.

ITEM NO.	ITEM	QUANTITY	UNIT OF MEASURE	UNIT PRICE	AMOUNT
				dollars	*dollars*
1	Clearing and grubbing		Lump sum	375.00	375.00
2	General excavation—unclassified	41,675.7	Cubic yard	1.40	58,345.98
3	Earth excavation in trenches	4,012.05	Cubic yard	1.00	4,012.05
4	Rock excavation in trenches	9,201.74	Cubic yard	4.00	36,806.96
5	Excavation and refill for future structures	2,264	Cubic yard	2.00	4,528.00
6	Rolled embankment	35,379.27	Cubic yard	0.60	21,227.56
7	Seeding and grassing	26,934	Square yard	0.06	1,616.04
8	Concrete in foundations 1:3:6	6,821.19	Cubic yard	8.00	54,569.52
9	Concrete in floors 1:2:4	3,721.95	Cubic yard	11.00	40,941.45
10	Concrete in structures 1:2:4	5,263.61	Cubic yard	17.00	89,481.37
11	Concrete in structures 1:2½:3	1,250.73	Cubic yard	17.00	21,262.41
12	Waterbound macadam road	7,912.89	Square yard	0.45	3,560.80
13	Concrete filter bottoms	8	Each	600.00	4,800.00
14	Concrete wash water troughs 1:1½:3	112	Each	16.00	1,792.00
15	Grout	1,773.11	Per bag	1.40	2,482.35
16	Concrete protection	454,347	Square foot	0.025	11,358.68
17	Reinforcing steel	1,413,630	Pounds	0.05	70,681.50
18	Structural steel	37,442	Pounds	0.06	2,246.52
19	Miscellaneous iron and steel	52,103	Pounds	0.18	9,378.54
20	Iron castings	29,219	Pounds	0.06	1,753.14
21	Cast iron B & S pipe 20″ and larger (delivered and installed)	523.18	Ton	52.00	27,205.36
22	Cast iron B & S pipe 20″ (delivered and installed)	17.826	Ton	50.00	891.30
23	Cast iron B & S specials (delivered and installed)	220.501	Ton	150.00	33,075.15

23	Cast iron B & S specials (delivered and installed)	220.501	Ton	150.00	33,075.15
24	Cast iron flanged pipe and specials (delivered and installed)	229.289	Ton	180.00	41,272.02
25	Placing castings, valves, gates, hydrants	276.596	Ton	62.00	17,148.95
26a	Cast iron culvert pipe (delivered)	778.176	Ton	43.50	33,850.66
26b	Cast iron culvert pipe (installed)	767.561	Ton	11.50	8,826.95
27	Wooden baffles in mixing basins		Lump sum	7,800.00	7,800.00
28	Setting acid, pressure and float tanks		Lump sum	250.00	250.00
29	Brickwork in baffles	66.78	Cubic yard	40.00	2,671.20
30	Brickwork for manholes and walls	70.33	Cubic yard	40.00	2,813.20
31	Dressed lumber in place	1.376	M. B. M.	180.00	247.68
32	Sheeting and shoring in place	3.061	M. B. M.	35.00	107.14
33	Six inch terra cotta pipe	379	Linear foot	0.39	147.81
34	Superstructures, buildings and appurtenances		Lump sum	246,152.40	246,152.40
35	Cleaning up		Lump sum	300.00	300.00

Total value of work 863,979.69
Extra work—$4,916.66

	AMOUNT
	dollars
Contract No. 13. Valves, hydrants, etc. at Filter Plant	47,966.25
Extra work—$28.35	
Contract No. 14. Plumbing	16,700.00
Extra work—$874.84	
Contract No. 15. Heating	27,500.00
Extra work—$418.00	
Contract No. 16. Electrical work	18,675.00
Extra work—$996.79	
Contract No. 17. Pumping equipment for Filter Plant: Opening of bids, March 5, 1930.	

TABLE 1—*Concluded*

ITEM NO.	ITEM	QUANTITY	UNIT OF MEASURE	UNIT PRICE	AMOUNT
				dollars	*dollars*
1	Motor driven centrifugal wash water pumps	2	Each	1630.00	3,260.00
2	Motor driven centrifugal flushing pump	1	Each	935.00	935.00
Total value of work					4,195.00
Extra work—none					
Contract No. 18. Filter equipment: Opening of bids—March 5, 1930.					
ITEM NO.	ITEM	QUANTITY	UNIT OF MEASURE	UNIT PRICE	AMOUNT
				dollars	*dollars*
1	60″ x 30″ Venturi meter		Lump sum	6,069.00	6,069.00
2	60″ x 30″ Low head Venturi meter		Lump sum	6,541.00	6,541.00
3	30″ Wash water meter and controller		Lump sum	4,077.00	4,077.00
4	Filter rate of flow controllers	8	Each	1,912.50	15,300.00
5	Water level controller		Lump sum	1,570.00	1,570.00
6	Operating tables	8	Each	612.00	4,896.00
7	Illuminated rate of wash gauge		Lump sum	607.00	607.00
8	Illuminated wash water tank level gauges		Lump sum	1,178.00	1,178.00
9	Automatic chlorine dosing equipment		Lump sum	7,550.00	7,550.00
10	Dry chemical feeding equipment		Lump sum	3,800.00	3,800.00
11	Acid storing and dosing equipment		Lump sum	1,372.00	1,372.00
12	Aerator nozzles	225	Each	28.80	6,480.00
13	Filter strainers	32,500	Each	0.32½	10,508.33
14	Gas generating plant		Lump sum	780.00	780.00
15	Refrigerator equipment		Lump sum	956.00	956.00
16	Laboratory incubator		Lump sum	1,371.00	1,371.00
17	Laboratory air compressor		Lump sum	246.00	246.00
18	Filter sand	1,039.19	Cubic yard	10.90	11,327.17
19	Filter gravel	623.52	Cubic yard	11.90	7,419.89
20	Operating and testing filter plant equipment		Lump sum	800.00	800.00
Total value of work					92,848.39
Extra work—$679.60					

level and is lifted to the second floor on a platform elevator which is 4 feet by 5 feet in size and which has a capacity of 2,500 pounds at a speed of 50 feet per minute. Two dry feed chemical machines are installed for the application of lime and these machines are of the same type and make as the alum machines. The eductors which are provided for handling the lime solution are connected to a system of piping which permits the application of this chemical either into the 60-inch influent main or into the 60-inch effluent main. Each alum machine has a range of capacity from 700 to 8,000 pounds per day and each lime machine has a range of capacity from 350 to 6,000 pounds per day. Both types of machines are guaranteed to be accurate within five per cent.

The sulphuric acid is stored in a steel tank located in a concrete pit outside of the main buildings. This tank is 6 feet in diameter by 24 feet long. Concentrated acid from this tank flows by gravity into two acid feeding tanks located in the basement of the chemical building. Each of these tanks is $3\frac{1}{2}$ feet in diameter by $4\frac{1}{2}$ feet high. The concentrated acid from these smaller tanks is delivered to an acid feed device which conveys the diluted acid to an eductor. This eductor is connected by a lead pipe to the 60-inch influent main at which point the acid will be applied.

The acid feed device which was manufactured and furnished by the American Water Softener Company, consists of two electro-magnetic proportioners, one for each of the acid feeding tanks, two lift pipes with balance weights and funnels to which are attached the eductors for forcing the dilute acid into the influent main. A small water meter is connected to the Venturi tube on the influent main which bypasses the incoming water by taking the water from the pressure side of the tube and discharging it slightly beyond the throat. The gear train of the water meter has quick make and break contacts which actuate either of the electro magnetic proportioners and with this meter connected to the Venturi tube, the amount of acid feed is at all times in direct proportion to the amount of water entering the plant. Each proportioner has an adjustable vernier so that it is possible to apply any desired amount of acid per thousand gallons of water treated. The funnel and connecting valves and piping which handle the dilute acid are made of hard rubber and the line transporting this dilute acid to the influent main is of lead.

The chlorine equipment consists of two automatic control solution feed vacuum chlorinators manufactured by Wallace and Tiernan

Company. Each of these machines has a maximum capacity of 100 pounds per 24 hours and is provided with chlorine scales with automatic recording gauges. The chlorine machines have an accuracy guaranteed within 4 per cent. In order to guard against interruption of service of these machines, due to a breakdown of the domestic water system, there is installed an auxiliary pump and pressure tank with a capacity to supply water to either machine at a rate of 8 to 10 gallons per minute against a pressure of 35 pounds. This system is automatic in operation and at any time that the pressure in the domestic system falls below 35 pounds, the auxiliary pump will start to operate. Space is provided on the first floor of the chemical building alongside of the chlorine machine rooms for the storage of flasks of chlorine.

OTHER MECHANICAL EQUIPMENT

As previously mentioned, a water level controller of the Builders Iron Foundry manufacture is installed to regulate automatically the quantity of raw water entering the filter plant. This regulation can be accomplished either by the level of the water in the aerator pool or by the level of the water in the filters. A float tank 40 inches in diameter and 9 feet high is installed in the basement of the chemical building. To this tank are connected pipes leading from the aerator pool and from the filters so that the water level at either of these two locations is transmitted to this tank. The water level controller is actuated by a differential, one side of which is connected to the float in the float tank and the other side of which is connected to the piston of the hydraulic cylinder on the butterfly valve in the influent main. A dropping of the water level on the filters causes the water level controller to operate a pilot valve which in turn causes the hydraulic cylinder to open the butterfly valve. A too rapid rise of the water level over the filters, causes the reverse operation. Likewise, a too rapid opening of the valve shuts off the pilot valve or a too rapid closing of the butterfly valve also shuts off the pilot valve. This water level controller operates very satisfactorily in maintaining a close range of level of water over the filters and maintaining a uniform rate of supply to the filter plant. A more detailed description of this apparatus will be found in the Journal of the American Water Works Association for October, 1932.

As previously mentioned, two large Venturi meters are installed at the filter plant, each of them 60 inches by 30 inches in size. These meters record the quantity of water entering and leaving the plant.

The registers are graduated to read up to 60 m.g.d., but are designed so that the reading can be changed at a later time to a maximum of 80 m.g.d. Each meter is capable of measuring accurately down to a rate of 10 m.g.d., has a guaranteed accuracy to within two per cent at 20 m.g.d. and to within one per cent at 80 m.g.d.

To facilitate the operation of the filter plant, illuminated gauges are provided for the convenience of the operator. At one end of the filter gallery, two gauges each 30 inches in diameter are installed. One gauge indicates the rate of washing the filters, the other gauge indicates the level of the water in the wash water tank. Within the pump room is a small illuminated gauge 10 inches in diameter, which indicates the level of the water in the wash water tank. This is for the convenience of the operator of the wash water pumps.

LABORATORY

The second floor of the laboratory building is provided with modern laboratory equipment, including the customary benches, cabinets, fume hoods, etc. It is desired as a matter of record to enumerate the following special items of equipment. In the kitchen is installed a General Electric refrigerator of a storage capacity of 45 cubic feet and with shelf area of 65 square feet. This refrigerator is guaranteed to maintain a temperature down to 40 degrees within an accuracy of 4 degrees. In the plating room is an incubator cabinet manufactured by Arthur H. Thomas Company with electric heating elements controlled by thermostat and solenoid to maintain a temperature 37 degrees C. In the laboratory storage room is a rotary air pump with a capacity of forty cubic inches of air per revolution and with a speed of 400 revolutions per minute. This air pump or compressor is Crowell type 2-D, manufactured by Arthur H. Thomas Company. On the first floor of the laboratory building is a gas generator for supplying gas to the laboratory. This generator is No. 1 E.M. manufactured by the American Heating and Lighting Company, with a capacity of 300 cubic feet per hour and supplying gas fuel containing 500 B.T.U. of heat per cubic foot of gas. The fuel used is absorption process gasoline of 86 to 88 degrees Beaume specific gravity.

The gas machine consists of an aerometer and blower which automatically supplies air under pressure to an evaporating storage tank or carburetor which contains the gasoline. The resultant gas passes through the mixer where it is mixed with air to produce gas of the proper heat value.

SUMMARY OF DESIGN DATA

The following briefly summarizes the main features of the filtration plant:

	m.g.d
Normal capacity (present construction)	32
Overload capacity (present construction)	40
Normal capacity (future development)	64
Overload capacity (future development)	80

Size of aerator basin—90 feet x 104 feet
5 rows of 44 nozzles—spaced 20¾ inches apart
Spacing of rows—15 feet apart
Capacity of present nozzle system—40 m.g.d. under a head of 11 feet
3 mixing basins— each 43 feet 6 inches x 37 feet 10 inches x 17 feet deep
Mixing basin detention period—20 minutes
Length of travel in mixing basins—1,200 feet
Velocity flow in mixing basins—0.9 foot per second
Loss of head in mixing basins—3 feet
3 coagulation basins each 50 feet x 202 feet 6 inches
Depth of coagulation basins varies from 18 feet at inlet to 10 feet at outlet
Coagulation basin detention period without sludge—2½ hours
Cogaulation basin detention period with allowance for sludge—2 hours
Coagulation basin velocity without sludge—1.32 feet per minute
Coagulation basin velocity with sludge—1.65 feet per minute
Velocity of basin, inlet channel—1.38 feet per second
Velocity of basin, outlet channel—1.00 foot per second
Filter unit capacity—4. m.g.d.
Number of Units—8.
Size of each unit—46.5 feet x 30.5 feet x 12.25 feet deep
Thickness of sand—30 inches
Thickness of gravel—18 inches
Normal rate of wash—24 inches per minute
Wash water storage—252,000 gallons
2 wash water tanks—each 31 feet 9 inches x 45 feet 6 inches x 11 feet 6 inches depth
2-3 m.g.d. wash water pumps—53 feet head
Elevation bottom wash water tanks above sand—26.25 feet.
Clear well, 2 basins—each 82 feet 3 inches x 166 feet x 11 feet deep
Clear well, capacity allowing 2 feet depth of lime sludge—2 m.g.
Length of 36-inch wash water drain—4,850 feet
Capacity of wash water drain—80 m.g.d.
Alum storage capacity—207.6 tons
3 Dry feed alum machines—Capacity of 700 to 8,000 pounds per day
2 dry feed hydrated lime machines—capacity 350 to 6,000 pounds per day
Lime storage capacity—74 tons
Storage sulphuric acid—5,000 gallons
2 automatic control solution vacuum feed chlorinators—capacity of chlorinators 100 pounds per 24 hours

(*Presented before the New York Section meeting, March, 22, 1925*)

SHORTAGE OF PUBLIC WATER SUPPLIES IN THE UNITED STATES DURING 1934

By Gilbert F. White

(National Resources Committee, Washington, D. C.)

Although precipitation was deficient to a greater extent than ever before during the spring and summer of 1934 in many parts of the United States west of the Appalachians, most of the public water supplies of the nation proved to be adequate. The abnormally small run-off and the recession of the shallow ground-water table, which caused severe shortage of water for domestic use and for livestock in numerous rural areas, resulted in general and acute shortage of public supplies in only a few sections of the country.

The conditions resulting from the drought focused attention on certain areas in which unfavorable geologic and hydrologic conditions hinder the development of supplies that are adequate during very dry years, and on specific municipalities which have failed to make efficient provision for the use of sources that are readily available.

In an effort to discover the areas of especially difficult environmental conditions, and the municipalities having deficient collecting systems, the Water Resources Section of the National Resources Board last autumn requested the aid of the American Water Works Association in determining the public supplies which suffered shortage during 1934.

SOURCES OF DATA

Tables 1 and 2 and figure 1 incorporate the returns from letters sent to all state sanitary engineers by the Association, and information from special reports prepared by several state planning board consultants and staff members. The state returns are not all comparable. The Kansas towns are only those in which water relief work was done by the state relief administration, the North Dakota towns are only those which appealed for aid to the state geologist, and no entries are made for Nevada in view of the fact that all public supplies there were reported as depleted and that no supporting information was furnished. In several states, notably Wyoming, the

TABLE 1

Public water supplies experiencing shortage during 1934

STATE	POPULATION CLASSES								TOTAL		PERCENT OF TOTAL POPULATION SERVED BY PUBLIC WATER SUPPLIES*
	250 to 1,000		1,000 to 5,000		5,000 to 10,000		More than 10,000				
	Number	Population	Number	Population	Number	Population	Number	Population	Number	Population	
Arizona	1	847	5	18,172					6	19,019	9.4
Arkansas			1	3,234	1	5,182	2	132,526	4	140,942	30.8
California	2	1,300	4	6,353	1	8,434	1	10,439	8	26,526	5.5
Colorado	2	1,182	2	4,124	1	8,665			5	13,971	2.2
Georgia	1	538	2	2,513					3	3,051	0.3
Idaho			6	15,487			1	18,337	7	33,824	17.9
Illinois			1	2,162					1	2,162	0.04
Indiana	1	877	4	6,334	1	5,709	4	96,114	10	109,034	5.5
Iowa	2	1,445	7	11,855	3	19,719			12	33,019	2.5
Kansas	6	3,736	9	20,763			1	12,243	16	36,742	3.9
Michigan					1	9,514			1	9,514	0.3
Minnesota			2	5,186	2	15,710			4	20,896	1.4
Missouri			19	44,081	2	11,402	1	13,875	22	69,358	3.4
Nebraska			1	4,930					1	4,930	0.8
Nevada					Insufficient data						
New Mexico	1	698	1	4,739					2	5,437	3.6
New York	11	7,060	12	26,231	2	13,602	2	332,570	27	379,463	3.1
North Carolina	2	1,500	6	8,815					8	10,315	1.1
North Dakota	13	7,963	6	9,213	2	14,638	2	45,731	23	77,545	43.5
Ohio	8	5,808	18	31,798	5	39,931	3	110,572	34	188,109	3.9
Oklahoma	14	8,137	12	26,738	3	20,459	2	40,498	31	95,832	9.1
Pennsylvania	3	2,250	7	15,800			1	19,300	11	37,350	0.6
South Dakota	1	690					1	12,000	2	12,690	5.1
Tennessee	2	1,469							2	1,469	0.1
Texas	9	6,471	2	5,658	10	24,319			21	36,448	0.1
Utah	65	32,243	33	61,622	1	5,282	1	130,948	100	230,095	60.7
Vermont	1	297	5	9,962			1	10,008	7	20,267	12.4
West Virginia			1	1,192					1	1,192	0.2
Wisconsin					1	7,394			1	7,394	0.4
Wyoming					Insufficient data						
Total	145	84,511	166	346,962	36	209,960	23	985,161	370	1,626,594	2.1

* Total population served by public water supplies taken from tabulation in Journal of American Water Works, January, 1935.

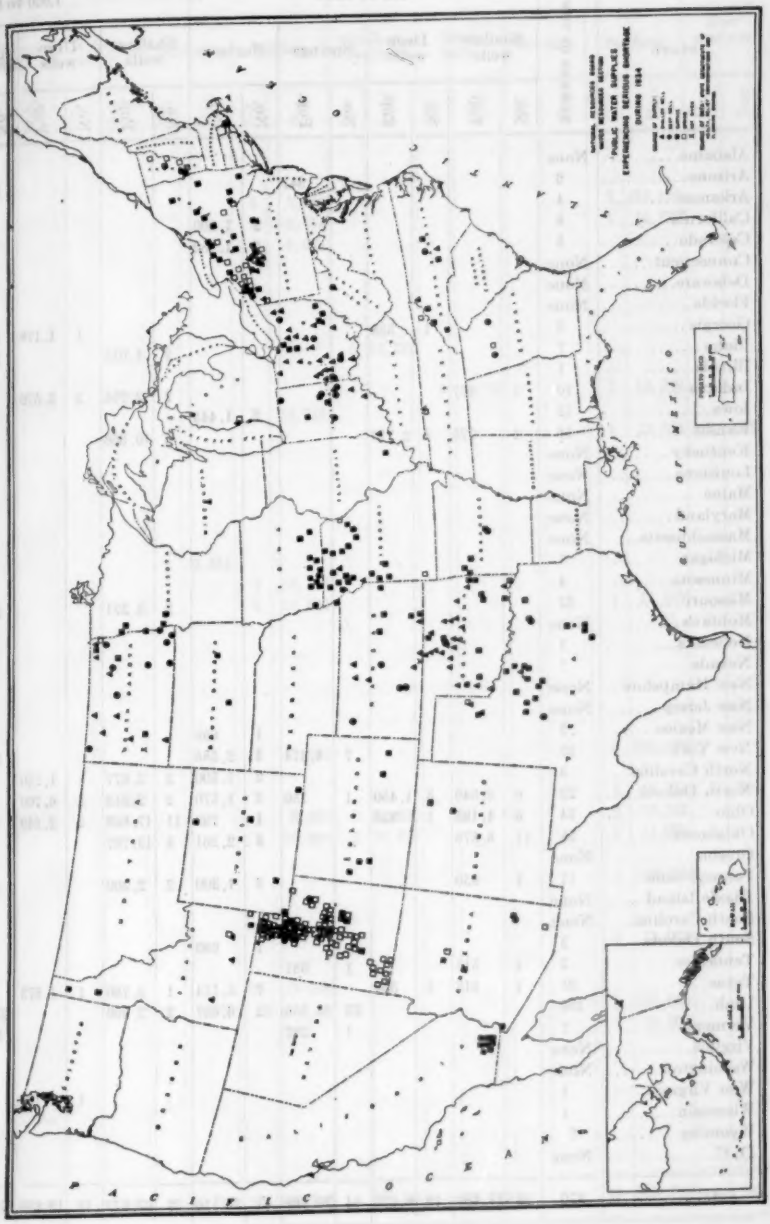

FIG. 1

Total.....	145	84,511	166	346,962	36	209,960	23	985,161	370	1,626,594	2.1

* Total population served by public water supplies taken from tabulation in Journal of American Water Works, January, 1935.

TABLE 2

Public water supplies experiencing shortage during 1934

STATE	NUMBER OF TOWNS	250 TO 1,000								1,000 TO 5,000							
		Shallow wells		Deep wells		Springs		Surface		Shallow wells		Deep wells		Springs		Surface	
		No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.
Alabama	None																
Arizona	6					1	847							1	3,891		14,291
Arkansas	4																3,234
California	8							2	1,300								6,353
Colorado	5							2	1,182								4,124
Connecticut	None																
Delaware	None																
Florida	None																
Georgia	3			1	538							1	1,178	1	1,335		
Idaho	7									2	4,016			1	3,235		8,294
Illinois	1																2,102
Indiana	10	1	877							2	2,764	2	3,570				
Iowa	12							2	1,445								11,855
Kansas	16	1	723	4	2,714					5	10,606						10,157
Kentucky	None																
Louisiana	None																
Maine	None																
Maryland	None																
Massachusetts	None																
Michigan	1																
Minnesota	4																5,186
Missouri	22									2	4,321			2	5,932		33,828
Montana	None																
Nebraska	1																4,930
Nevada	?																
New Hampshire	None																
New Jersey	None																
New Mexico	2							1	698								4,719
New York	27					7	4,474	4	2,586					2	2,720		23,511
North Carolina	8							2	1,500	2	2,677	1	1,730				4,408
North Dakota	23	6	3,540	3	1,450	1	750	2	1,570	2	2,513	4	6,700				
Ohio	34	6	4,195	1	830			1	783	11	17,868	2	2,140				11,700
Oklahoma	31	11	5,876					3	2,261	8	13,767						12,951
Oregon	None																
Pennsylvania	11	1	950					2	1,300	2	2,300			1	1,800		11,700
Rhode Island	None																
South Carolina	None																
South Dakota	2							1	690								
Tennessee	2	1	515			1	981										
Texas	21	1	813	1	544			7	5,114	1	3,780	1	1,878				
Utah	100					53	25,556	12	6,687	2	2,959			23	46,210		12,453
Vermont	7					1	297							3	4,652		5,310
Virginia	None																
Washington	None																
West Virginia	1											1	1,192				
Wisconsin	1																
Wyoming	?																
D. C.	None																
Total	370	28	17,489	10	6,076	64	32,868	41	27,116	39	67,571	12	18,388	34	69,775		191,238

TABLE 2

shortage during 1934 classified by size of population and source

	5,000 TO 10,000								MORE THAN 10,000								SOURCE NOT KNOWN	
	Shallow wells		Deep wells		Springs		Surface		Shallow wells		Deep wells		Springs		Surface			
	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.	No.	Pop.
							1	5,182							2	132,526		
							1	8,434							1	10,439		
							1	8,665										
									1	18,337								
			1	5,709							1	12,795			3	83,319		
							3	19,719										
															1	12,243	1	299
					1	9,514												
							2	15,710										
							2	11,402							1	13,875		
							2	13,602							2	332,570		
			2	14,638											2	45,731	1	653
4	31,175						1	8,756	1	31,820					2	78,752		
							3	20,459	1	26,399					1	14,099		
															1	19,300		
															1	12,000		
							10	24,319										
					1	5,282									1	130,948		
															1	10,008		
1	7,394																	
5	38,569	3	20,347	2	14,796	26	136,248	3	76,556	1	12,795			19	895,810	2	952	

authorities consulted, lacking requisite facilities for collecting precise information on the adequacy or inadequacy of supplies, submitted only rough estimates. It was also impossible to check the sources of supply in all cases. There appears to have been some difference in opinion from state to state as to the meaning of "serious shortage." In the final compilation an effort was made to include only those supplies which failed completely or which were in such danger of failing that special emergency measures were taken. Wherever possible, supplies reported as presenting minor control problems of taste, odor, and the like, or from which the use of water for sprinkling purposes was restricted for short periods were excluded.

Notwithstanding certain variations in the data obtained, it is believed that the returns furnish a moderately dependable basis for making generalizations with respect to the distribution of shortage in the United States as a whole.

PROBLEM AREAS

The conditions of precipitation, ground-water, and run-off in 1934 seem to have caused acute problems of water supply in eight major areas and in a number of lesser areas. Within each of these problem areas the difficulties encountered in securing adequate supplies in both normal and drought years are somewhat similar. Throughout each, ground and surface water conditions are roughly similar, and the major ground-water provinces as delimited by O. E. Meinzer (1) have been used to distinguish most of the boundaries. In some the shortages affected only a small proportion of the population served by public supplies, while in others the shortage was felt by all or nearly all of the urban population.

The outstanding characteristics of each problem area are summarized briefly in the following paragraphs.

The lower Colorado Basin area

No other section of the United States experienced as nearly complete a failure of public water supplies as did the lowermost section of the Colorado Basin. The flow of the Colorado River in its lower course was smaller during the summer than in any earlier time of record (2), and as a consequence, Yuma, Arizona, and eight towns in the Imperial Valley of California, all of whom secure their supplies from the Colorado, suffered shortage. Although the entire flow of the river was diverted for irrigation and water-supply purposes, it

became necessary during July and August to ship water into the Imperial Valley by tank car. Fortunately, a recurrence of these conditions will be precluded by the Boulder Dam which will insure a minimum flow sufficient to meet abundantly the water requirements of the Valley.

Colorado Plateaus area

A relatively large proportion of the towns in the sparsely populated Colorado Plateaus area of northern Arizona, eastern Utah, western Colorado, and southwestern Wyoming were faced with shortage. The water supplies for the few towns in this arid section are secured chiefly from streams, springs and deep wells. The cumulative deficiency of snowfall on the Southern Rockies resulted in a very small run-off during the spring and early summer of 1934, and the surface supplies at a number of places on the upper tributaries of the Colorado River, notably Grand Junction, Flagstaff, and Williams, proved insufficient to meet normal consumptive demands through the summer. The springs supplying several smaller towns also failed. For the larger towns the chief solution seems to lie in a provision of additional storage capacity or in the utilization of ground waters, which vary greatly in accessibility in different parts of the plateaus.

The Great Basin area

Shortage apparently affected a higher percentage of the urban population of Utah than of any other state. With the exception of difficulties resulting from a few inadequate surface supplies which have been mentioned in connection with the Colorado Plateaus area, the chief trouble developed in the western or Great Basin half of the state where most of the municipalities are located in alluvial valley bottoms. Approximately 50 villages, ranging in population from 250 to 1,000, and 20 towns, of from 1,000 to 5,000 population, were inconvenienced by the failure of springs. In addition, the surface supplies for about 10 villages and an equal number of towns became seriously depleted. Adequate supplies for many of those towns probably can be secured from the alluvial materials filling the valleys between the mountains. It appears, then, that a repetition of the 1934 type of shortage could in general be prevented by relatively cheap well drilling operations under the advice of competent geologists. A case in point is the Salt Lake City supply, which was derived from rapidly failing and expensive surface sources until the

U. S. Geological Survey experts pointed out the existence of a nearby underground reservoir. Wells drilled into this reservoir yielded sufficient water to supplement adequately the existing surface supply. There are insufficient data to warrant any generalizations with respect to the scattered Nevada communities sharing the same type of environment as western Utah.

Southwest Permian and Pennsylvania area

The regions in which the relatively impervious rocks of Permian and Pennsylvania age lie at or near the surface in west-central Missouri, southeastern Kansas, Oklahoma, north-central Texas, and west-central Arkansas constitute the fourth important area of shortage. Throughout most of this area potable supplies in deep lying aquifers are lacking, and the shallow ground-water supplies are meagre except in valley fills. Public supplies are derived chiefly from large streams, reservoirs on small streams, and shallow wells. A number of the larger streams had but slight discharge in 1934 and carried a heavy concentration of minerals during the summer months. Even Little Rock, Arkansas, which obtains most of its supply of water in normal times from the Arkansas River was forced to make over-drafts on its supplemental wells in an effort to minimize the use of the river water, which because of the low flow, had an extremely high content of sodium chloride. Communities utilizing lakes and reservoirs were most seriously affected. Storage for small and medium-sized towns was depleted greatly from Missouri to Texas. Complete failures occurred at three communities on the Permian Plains of Texas, and at one community in the Flint Hills section of eastern Kansas. Depletion of shallow wells supplies was restricted for the most part to communities located in the valley bottoms of central Oklahoma. Enid was the largest of these. In view of the slight importance of deep ground sources, the major practicable means of securing additional supplies seem to comprise the supplementing of reservoir and channel storage capacity, and the further development of natural underground storage where conditions permit. As Earnest Boyce has suggested, this may require joint planning with respect to surface works by groups of communities concerned with water supply, sewage disposal, and flood control on the same stream (3). The Grand-Neoshc Basin in southeastern Kansas and eastern Oklahoma furnishes an excellent example of the need for a coördinated development of surface and ground waters in relation to other uses as well as public supply (4).

Kansan drift area

The failure or near-failure of impounding reservoirs supplying moderately large towns was a common source of complaint in the areas in southern Iowa, northern Missouri, and southeastern Nebraska which are covered by Kansan drift. The Kansan drift is relatively thin and clayey, and contains few water-bearing sands and gravels, while most of the deep-lying waters are highly mineralized. Surface sources have therefore been utilized in most places. No less than 20 of these proved inadequate during the summer. The causes of the failure of the southern Iowa supplies as discussed by A. H. Wieters (5) seem to apply throughout the area. In general, storage capacities were too small to supply demands for more than a year. In all but a few cases an addition to the existing reservoir, or, where a river is used, a new reservoir, would provide abundant supplies even should there be a recurrence of 1934 conditions. Such work is expensive, however, and the drilling of deep wells to aquifers which careful explorations show to be suitable sources of auxiliary supplies may be the most practicable solution. Given an adequate amount of hydrologic and geologic data, the problems are simpler than those in the Southwest Permian and Pennsylvanian area.

Dakota drift and Red River Valley areas

The drought served to accentuate several recurring questions concerning sources of public water supply in the drift-covered parts of the two Dakotas and in the Valley of the Red River of the North.

The sections of the two states lying north and east of the Missouri River are not only covered with glacial drift which is water-bearing in most places, and especially in valleys, but also are underlain by Dakota sandstone (the waters in much of which are under artesian pressure). Large towns have found the artesian waters unsatisfactory in quality and quantity, and many of them have turned to shallow or surface sources. Several of the North Dakota shortages may be attributed largely to the abnormally high summer demands on deep wells that for years had been proving less and less dependable, in part because of the deterioration of casings and screens. Some of the towns that had previously constructed reservoirs, such as Mitchell, South Dakota, managed to pass through the summer without shortage, while others, such as Huron, South Dakota, found their channel storage in the main streams to be inadequate and were forced hastily to tap shallow waters. On the other hand, a considerable

number of North Dakota communities having shallow wells were faced with shortage resulting from a recession of the shallow ground-water table that is reported to have amounted in some sections to as much as 20 feet during recent years. A consideration of remedial measures therefore hinges on the relative cost of securing additional supplies from ground sources and, in part surface sources.

It is in connection with the evaluation of the surface sources that the Red River Valley problems merit consideration with the Dakota Drift problems. The lacustrine silts of that valley are poor water-bearing formations, and the artesian waters in the western half of the valley are highly mineralized (6 and 7). The major cities, including Grand Forks and Fargo, draw their supplies from the Red River of the North. Run-off in the drift-covered portions of the drainage basin to the west as well as in the northwestern Minnesota portions to the east was extraordinarily light in 1934, and flow in the main stem ceased entirely for a short time during the summer. On account of the normally meagre run-off and the inadequate and dwindling ground waters, the regulation of the stream flow in the Red River Basin, including a part of the Dakota Drift area, has become increasingly urgent, and was never more needed than during 1934.

Indiana—Ohio—New York drift area

The drought was reported to have been partly responsible for shortage in more than 80 towns in the drift-covered sections of Indiana, Ohio, Pennsylvania and New York. In Indiana, emergency preparations were made at Muncie, Bloomington, and Logansport to supplement the regular surface supplies. Lesser taste and odor problems arose in several neighboring cities using surface supplies from the White River system, and special chemical control was necessary at Anderson and Ft. Wayne. Other large towns and cities having difficulties with surface sources were Bucyrus, Delaware, and Springfield in Ohio, and Bradford in Pennsylvania. It is significant that, although ground-water supplies in Ohio were not greatly depleted during the drought of 1930–1931, a number of towns and cities in that state, notably Mansfield, experienced a shortage of their ground-water supplies during 1934, apparently because of a protracted deficiency in precipitation that began as early as 1930 (8). It is also probable, as W. L. Stevenson suggests, that the storage capacities of many of the water works in the Ohio Basin were increased after the 1930–31 drought, or that the emergency supplies developed at

that time were continued in service. Most of the shortages reported in New York were in small towns depending upon surface sources. These communities, like the city of Rochester, which was reported to have had some deficiency, were embarrassed not by any lack of available water, but by their earlier failure to extend inadequate or expensive collecting systems that had been in need of improvement for several years. In the area as a whole, the shortages in small towns depending on ground sources cannot be regarded as serious, for there is an abundance of ground water in most parts of the drift. However, the deficient supplies from certain surface sources indicate the need for coördinated investigation of potential underground storage and of regulation of streams and perhaps of entire basins for water supply and pollution-control purposes. The conditions of 1934 have called attention to at least one basin, that of the White River, in which deficient supplies combined with waste disposal problems appear to call for unified consideration in the near future.

Minor problem areas

Of the less prominent areas of shortage, one of the better defined comprises an area on the upper border of the coastal plain in eastern Texas and southeastern Oklahoma where the Upper Cretaceous materials lying on the surface yield water that is hard and brackish. Surficial water-bearing sands are of restricted distribution and underground aquifers lie at a depth of several thousand feet, so that surface sources have been developed by almost all municipalities. The extreme drought, coming as a rare occurrence, caused the failure or near exhaustion of at least nine of these supplies.

It is notable that distress was not common in the Great Plains country. A few shallow well supplies in western Kansas were depleted to a point necessitating the emergency construction of supplemental wells, and about six surface supplies on the eastern slopes of the Rockies were reported as having shortage. This situation appears to have been due in part to availability of water, and in part to the fact that the relatively sparse urban population had made adequate adjustments to the frequent occurrence of arid weather conditions in this semi-arid province. Thus, in the Pierre Shale area of western South Dakota, where water is at premium in normal times and where the precipitation deficiency was very great during 1934, the few water works were not faced with unusual difficulties.

Other minor groups of public supplies experiencing shortage in-

cluded scattered municipalities on the Snake River plains, and in northern Vermont.

SUPPLIES HAVING CHRONIC SHORTAGE

The shortages shown in Tennessee, North Carolina, Georgia, Michigan, and Wisconsin seem to have been the consequence of a failure to maintain wells or to build suitable impounding works rather than the consequence of drought. In the opinion of the respective state sanitary engineers, the shortages were due to inadequate plants that deserve corrective work if satisfactory service is to be given in most years.

It is probable that an extension of the drought to other parts of the country in future years would emphasize the existence of other problem areas, and it is possible that a continuation of the recent drought might cause widespread and serious shortage in areas that experienced only slight difficulties during 1934. However, from the standpoint of quantity, the events of last year have demonstrated for a large portion of the country the general adequacy of supplies and, at the same time, the existence of special problem areas.

SUMMARY

1. Notwithstanding the severe and widespread nature of the drought of 1934, relatively few public water supplies in the United States experienced shortage. Approximately two percent of the total population served by public water supplies was affected.

2. The chief areas in which there were serious shortages of water were: *1*, the Lower Colorado Basin; *2*, the Colorado Plateaus; *3*, the Great Basin; *4*, the Southwest Permian and Pennsylvanian area; *5*, the Kansan Drift area; *6*, the Dakota Drift area; *7*, the Red River Valley of the North; and *8*, the Indiana-Ohio-New York Drift area.

3. Minor shortages due in large measure to the drought occurred on the upper coastal plain of Texas, the Snake River plains, along the east slope of the Rocky Mountains, in western Kansas, and in Vermont.

4. Shortages also were experienced by certain supplies in Wisconsin, Michigan, New York, Tennessee, Georgia, and North Carolina which have chronic deficiency.

5. The public supplies of communities ranging in population from 250 to 5,000 constituted the greater proportion of those adversely affected. In this class the sources were about evenly divided among

shallow wells, springs, and surface sources. In communities having populations of more than 5,000 each, surface waters were the major sources of supply in all but thirteen cases.

CONCLUSIONS

1. From the standpoint of quantity of water, the public water supplies in many areas affected by the drought appear to have been adequate during 1934. This reflects general credit on the engineers and geologists responsible for the location and design of collecting systems and on the managers who maintained their services without marked deficiencies in quantity during drought. There are, however, a number of areas in which the public has faced shortage problems.

2. In two of the major problem areas—the Red River of the North and the Southwest Pennsylvanian and Permian areas—the environmental conditions make difficult the development of public water supplies which are adequate in normal years, let alone in abnormal years of drought. Careful geological and engineering surveys involving groups of communities as well as individual communities will probably be necessary if satisfactory supplies are to be secured throughout those areas.

3. In the remaining six major problem areas the shortages, although severe in 1934, seem for the most part to be preventable in the future by the completion of work now in progress or by the construction of relatively small additional works. Satisfactory solution may require the improvement of storage facilities in accordance with lessons learned from the precipitation, run-off, and evaporation extremes of past years as modified by the 1934 experiences. It may further require an extensive exploration of ground-water resources, and in some cases it may necessitate a regional rather than a local approach to water supply and sanitation problems.

REFERENCES

The data used in this study are taken chiefly from letters from the respective state sanitary engineers, and from the planning board staffs in Ohio and Texas.

(1) MEINZER, O. E.: "The occurrence of ground water in the United States." U. S. G. S. *Water-Supply Paper*, No. 489.

(2) HOYT, JOHN C.: "Drought, weather and streamflow." *Eng. News-Record*, 114: 199–200.

(3) BOYCE, EARNEST: "Public and emergency water supplies during the drought period in Kansas." *Jour. Amer. Water Works Assoc.*, 27: 164–173.
(4) P. W. A., Mississippi Valley Committee, *Report*, p. 192.
(5) WIETERS, A. H.: "The effect of the drought on public water supplies in Iowa." *Jour. Amer. Water Works Assoc.*, 27: 154–163.
(6) TARBELL, W. P.: "Red River Valley water problems." *Jour. Amer. Water Works Assoc.*, 26: 301–308.
(7) LIUM, E. L.: "Experiences with treating dwindling water supplies in northwestern Minnesota." *Jour. Amer. Water Works Assoc.*, 26: 194–200.
(8) WARING, F. H.: "Water supply problems in Ohio occasioned by the drought." *Jour. Amer. Water Works Assoc.*, 23: 793–800.

NEW WATER SOFTENING PLANT AND CLEANING EXISTING WATER MAINS AT WOODSTOCK, ILLINOIS

By John A. Fulkman

(Hydraulic Engineer, Consoer, Townsend & Quinlan, Consulting Engineers, Chicago, Ill.)

Woodstock has a population of 5500, and is located about 50 miles north west of Chicago. It is the county seat of McHenry County, a farming district.

The water consumption is mostly domestic with the usual commercial consumption, and an industrial consumption of about 50,000 gallons per day. The industrial consumption is mostly taken by the Borden Dairy Company and The Woodstock Typewriter Company. The average water pumped is 526,000, and the maximum about 800,000 gallons per day.

The water supply is secured from two gravel packed wells, each 200 feet deep. These wells were drilled in 1926. Recent tests show that 1000 gallons per minute can be pumped from one, and 900 gallons per minute from the other well, to the ground surface. The recent dry years do not seem to have affected the production of the wells. The additional head required for pumping water to the softening plant reduces the quantity pumped from the wells, with the existing deep well pumps, to 850 and 750 gallons per minute, respectively. The wells are about 150 feet apart.

CHARACTER OF WATER

The water is the typical hard water, with considerable iron, found in well supplies in Illinois and surrounding states. Therefore, softening and iron removal are desirable, economical, and practicable.

The total hardness of the Woodstock water is 360 and the iron content 1.25 p.p.m. The hardness consists of about equal amounts of calcium and magnesium carbonates. The water also contains some hydrogen sulphide.

DESCRIPTION OF PROJECT

The Woodstock water project, which has been completed during the past year consists of a million gallon per day softening and iron

removal plant, and cleaning 84,000 feet of 4-, 6- and 8-inch old cast iron water pipe. A new concrete cover was also placed on one of the existing reservoirs.

The work was done as a Federal Public Works Project. The application for the loan and grant was made in October, 1933; bids were received in May, 1934; construction started August, 1934; and the work was completed and the softening and iron removal plant placed in operation in January, 1935.

The softening and iron removal plant has a nominal capacity of one million gallons per day. The average water consumption is 526,000 gallons per day and therefore the plant is only operated about half of the time. The cycles between operations are 3 or 4 hours.

Either of the wells will supply over a million gallons per day rate and, therefore, only one well is in operation, when operating the plant.

The water treatment consists of aeration, chemical mixing, settling, carbonation, filtration, and chlorination. A lime-soda ash type of plant is provided. However, only lime is being used. All of the tanks and basins for the plant were built of concrete.

Wells

The water is pumped from the wells by the existing turbine pumps. The well water is carried to the softening and iron removal plant through new cast iron lines connected to the existing lines leading from the wells to the existing reservoirs. The existing pipe connections from the well lines to the reservoirs were left in place and valved, so that water from the wells can by-pass the softening and iron removal plant in case it is ever necessary on account of a bad fire.

The water from the well in use when operating the softening and iron removal plant is, under ordinary operation, carried to the aerator. However, a second connection is made so if it is desirable to by-pass the aerator, the well water can be carried directly to the mixing chamber.

Aerator

The aerator is installed in a concrete aeration chamber 12 feet by 6 inches square and about 12 feet deep, located in the filter and chemical house. Lime can be applied in a trough under the aerator discharge plate, and mixed by the agitation of the aerated water. A

drain is provided in the aeration chamber, as well as a 2½-inch pressure line connection for cleaning.

Mixing chamber

The water from the aeration chamber flows by gravity to the mixing chamber. The mixing chamber is 12 feet by 6 inches square and about 12 feet deep. Mechanical mixing is provided. This mixing chamber has a capacity of about 14,000 gallons which will give a 20 minute mixing period at a one million gallon per day rate. Provision is made for applying lime in the mixing chamber, as well as in the aeration chamber, and for applying a coagulant in the mixing chamber. The water from the mixing chamber flows to the clarifier.

Clarifier

The mixed water is carried to the influent end of the clarifier where it is distributed by a wooden slotted baffle. The wood baffle has vertical slots 1 inch wide every 8 inches. These slots extend from 18 inches below the top of the clarifier walls down 11 feet or within 2½ feet of the bottom. The water is taken from the clarifier at the effluent end by a weir trough near the top. Provision is also made for applying a coagulant as the water leaves the clarifier.

The clarifier is 30 feet square at the top, 28 feet square at the bottom, with side walls 15 feet deep. Considering one foot free-board or a water depth of 14 feet, the clarifier has a capacity of 89,000 gallons. This gives slightly over two hours retention period.

Mechanical sludge removal equipment is provided in the clarifier. A drain is provided from the clarifier which is discharged into a nearby storm water drain. A 2½-inch pressure hose connection is provided in the clarifier for cleaning.

The water from the clarifier is taken under ordinary operation to the settling basin. However, provision is made for taking the water from the clarifier to the carbonation basin, thus by-passing the settling basin, or providing for carbonation before the water passes through the settling basin. The water can also be taken from the clarifier direct to the filters.

Settling basin

The settling basin is the same size and shape as the clarifier, thus giving slightly over two hours settling period. The settling basin has no mechanical sludge removal equipment. It is designed so

that, if it is ever desirable to install such equipment in the basin, it can be done. The same design of baffle and effluent trough is provided in the settling basin as in the clarifier. The settling basin is provided with drain and pressure line for cleaning.

The water from the settling basin can be taken to the carbonation basin or direct to the filters if desired.

Carbonation basin

The settled water is conveyed under ordinary operation to the carbonation basin. However, as mentioned before, the water from either the settling basin or the clarifier can be taken directly to the filters if desired.

The carbonation basin has an area of 15 feet by 18 feet and 14 feet water depth. This gives a capacity of 28,000 gallons which is about 40 minutes retention period.

The carbon dioxide is generated in a coke burner in the boiler room of the power plant, scrubbed and conveyed to the carbonation basin. The carbon dioxide is distributed in the carbonation basin by a grid of wrought iron pipe in the bottom of the basin.

The water is conveyed through a 12-inch pipe from the carbonation basin to the filters. If desired, as mentioned above, the water can be taken from the carbonation basin back to the settling basin, and then to the filters.

Filters

Two gravity concrete rapid sand filters of 500,000 gallons per day capacity each are provided. Each of these filters has an area of 12 feet by 15 feet. The filters are provided with controllers, rate of flow and loss of head gauges and hydraulic operated valves.

The filters are washed by a 2800 gallon per minute wash water pump taking its suction from the filtered water reservoir below. This gives a wash water rate of 24-inch rise per minute. The wash water is taken from the filters to a storm water sewer.

The filter material consists of 30 inches of filter sand having an effective size of 0.44 m.m. and a uniformity coefficient of 1.5, and 18 inches of gravel graded as follows:

Depth, inches	*Size, inches*
5	$1\frac{1}{4}$ to 1
4	1 to $\frac{5}{8}$
4	$\frac{5}{8}$ to $\frac{3}{8}$
3	$\frac{3}{8}$ to $\frac{1}{8}$
2	$\frac{1}{8}$ to $\frac{1}{32}$

The under drainage system of the filters consists of a concrete box 16 inches square placed in the center of the filter under the filter floor. Cast iron laterals are connected to this concrete box every $6\frac{7}{8}$ inches. The laterals are $2\frac{1}{2}$ inch inside diameter with $\frac{1}{4}$-inch holes spaced every 2 inches pointing downward at 45° angles. Also each lateral has four $\frac{1}{4}$-inch holes at the end next to the filter wall. Each filter is provided with two concrete wash water troughs.

Filtered water reservoirs

Two existing reservoirs were used for filter water storage. One of these is a rectangular reservoir 28 feet by 56 feet by 12 feet deep, over which the new filter and chemical house was built. The capacity of this rectangular reservoir is 118,500 gallons. The other reservoir is a circular reservoir about 42 feet diameter with 10 feet depth. The circular reservoir has a capacity of about 94,000 gallons. A new concrete cover was placed on this circular reservoir.

Both of these reservoirs will be used for filtered water storage giving a total capacity of 212,500 gallons. The high lift pumps take their suction from these two reservoirs at the opposite sides from which the filter water discharges into the reservoirs.

The reservoirs are cross connected. Either of these reservoirs may be taken out of service for inspection or cleaning, while the softening and iron removal plant is in service.

Chlorination

A chlorinator is provided so that the water is chlorinated as it passes from the filters to the filtered water reservoir. From 0.1 to 0.2 p.p.m. residual chlorine is maintained.

Filter and chemical house

The pipe gallery floor is at about ground level. The pipe gallery is well lighted by windows, is well ventilated and readily accessible. Provision is made for entering this building at the north, east, and south sides.

The floor above the ground floor is for filter operation, laboratory and for housing the aeration chamber, mixing chamber, and dry feed machines. The laboratory is partitioned off from the rest of this floor. Also the aeration chamber and mixing chamber are partitioned off from the filter operating floor.

The chemical storage floor is above the aeration chamber, mixing

chamber, laboratory and part of the filter operating floor and is about 26 feet by 26 feet. The chemicals are conveyed from an automobile truck to the chemical storage floor by an electric trolley hoist. The hoppers from the dry feed machines extend to the chemical storage floor.

Access is provided to the walkways over the clarifier and settling basin and carbonation basin from the floor over the aeration chamber, and mixing chamber.

CLEANING EXISTING WATER MAINS

Considerable difficulty has been experienced in the past with red and black water. Therefore, it was decided to clean the existing cast iron water mains. All cast iron water lines in the City, with the exception of those laid within the last few years, were cleaned, aggregating 84083 feet of 4, 6 and 8 inch mains.

The water lines were cleaned by the National Water Main Cleaning Company. All 4 and 6-inch lines were cleaned dy dragging the sheet metal scraping tool through the water pipe with a $\frac{5}{8}$-inch diameter cable and a wench.

The $\frac{5}{8}$-inch cable was first dragged through the section of main being cleaned by a cable about the diameter of a lead pencil. This small cable was inserted into the pipe by attaching it to a stiff rubber cone the diameter of the pipe, and forcing the rubber cone through the pipe by water pressure.

The scraping tool consisted of stiff sheet metal prongs about $1\frac{1}{2}$-inch wide and bent in such shape as to spring a sharp scraping edge against the inside of the pipe. There were many of these prongs lashed together to form a unit of the scraping tool. The scraping edges of the prongs were lapped, staggered and one placed behind the other so as to scrape all surfaces of the inside of the pipe several times. Several such scraping units were tied together with short lengths of steel cable, so that one unit would drag behind the other when cleaning the pipe.

The cleaning tool was dragged or driven through the pipe, and the cleaning was done by scraping the inside surface of the pipe, and not by any revolving cutting tool. The majority of the 8-inch lines were cleaned by driving the cleaning tool through the pipe by the city water pressure. In all cases the city water pressure was turned on back of the cleaning tool, and the cleanings washed ahead of the cleaning tool to the outlet end of the section of pipe being cleaned.

The pipe was cleaned in sections of from a few hundred up to 2000 feet. After cleaning a section of main it was sterilized with one ounce of chloride of lime per 500 gallons of water before being placed back in service.

The material removed was mostly a fine material similar to the solids removed when flushing rusty mains, but it was black instead of brown. There was little scale on the pipe before cleaning, but there was a coating all around the pipe of this black slimy material which was not removed by flushing. This black material may have been iron deposits blackened by the presence of some hydrogen sulphide in the water.

The benefits in cleaning were elimination of black and rusty water, and an increase of pressures and quantities of water at various points on the distribution system.

Before the mains were cleaned it required 70 pounds pressure at the pumping station, to have the standpipe full. This standpipe is located about a mile from the pumping station. After cleaning the mains it only required 65 pounds pressure at the pumping station, to keep a full standpipe.

Pressure and flow tests were taken at fire hydrants at various points on the distribution system before and after cleaning the mains. At seven typical points on the distribution system the average flow at fire hydrants was 9 per cent greater after cleaning the mains. This was effected with a 5 pounds lower pressure at the pumping station. This 5 pounds saving in pressure will effect an annual savings in pumping of at least $300.

OPERATING SOFTENING AND IRON REMOVAL PLANT

The softening and iron removal plant has now been in operation only three months, and it is therefore still in the experimental operation stage.

For about a month after starting the plant was operated by taking the water through the aerator, mixing chamber, clarifier, settling basin, carbonation basin and filters. The chemical dose was 2½ pounds of hydrated lime per 1000 gallons, and at times 0.7 g.p.g. of alum. The lime was added in the aeration chamber under the aerator discharge plate, and the alum in the mixing chamber at certain times and at other times to the water as it left the clarifier. The hardness was reduced from 360 to 220 p.p.m. Under this operation

there seemed to be considerable precipitate carried over to the filters. There was also an excess deposit in the aeration chamber.

During the second month of operation the water was passed through the plant the same as during the first month. The chemical dose was 3.8 pounds of hydrated lime per 1000 gallons applied in the aeration chamber, and 0.45 g.p.g. of sodium aluminate added in the mixing chamber. The hardness of the water was reduced from 360 to 120 p.p.m. Under this operation less difficulty was experienced with precipitate being carried over to the filters. However, the deposit in the aeration chamber still continued.

During the last three weeks of operation, the plant has been operated by by-passing the aerator and aeration chamber. Under this operation the raw well water is taken direct to the mixing chamber, and the hydrated lime and sodium aluminate added in the mixing chamber. The chemical dose is 3.8 pounds of hydrated lime per 1000 gallons and 0.45 g.p.g. of sodium aluminate. This method of operation eliminates the additional head necessary to pump the water through the aerator. The water is softened by this method from 360 to 100 p.p.m. There has been no deposit of sediment in the mixing chamber.

I have suggested to Mr. C. K. Austin the chief engineer at the plant, that he try operating the plant by taking water through the aerator and aerator chamber, adding the lime and sodium aluminate in the mixing chamber, that he reduce the amount of lime, by the amount it takes to precipitate the iron and free carbon dioxide, and compare the results and operating cost of this method with the present method of by-passing the aerator.

When operating the aerator the water was practically saturated with oxygen. It is feared that a water with practically saturated oxygen may cause corrosion in the mains. However, the marble dust test with aerated water shows that the water is practically neutral and will not cause corrosion or deposit in the mains.

The pH of the raw water is about 8 and the treated water from 9.0 to 9.6.

BENEFITS OF SOFT WATER

The comments on the benefits of soft water have been very favorable. The consumers like the water for drinking and the soap saving is an outstanding fact.

One restaurant estimates they will save $50.00 a year in soap compounds, for dish washing alone; and another restaurant estimates their soap savings for dish washing at $35.00 a year.

The Almite Die Casting Company used about $3000 worth of water a year up to 1930. In 1930 they started using their own well and their yearly bill dropped, for the last four years, to $40.00 or $50.00. It is reported that since the soft water has been furnished this Company has started using all city water again. The local ice company also used about $1200.00 worth of city water a year, up to a few years ago when they drilled their own well. It is reported that the ice company, also, is using city water since it has been softened.

The water works operators at Woodstock estimate they used from 10 to 15 percent of the water pumped, for flushing mains to remove iron, before the softening and iron removal plant was in operation. Now they use practically no water for flushing mains.

After deducting the PWA grant the softening and iron removal plant with all overhead charges will cost the City about $35000. Bonds running from 1 to 25 years have been issued to take care of this obligation. The financial charges, including interest and amortization of these bonds, average $2250.00 per year. The annual cost of chemicals will be about $5000.00 and miscellaneous expenses $430.00 a year. This makes a total cost of $7680.00 per year for softening, or an average of 4 cents per 1000 gallons. Comparing this cost another way it is $1.40 per capita per year. The soap savings will be at least $3.00 per capita per year or a net saving of $1.60 per year per capita. This investment of $35,000 in the softening plant pays the citizens of Woodstock a net dividend of $8800.00 or 25 percent a year on the investment. This is considering soap savings alone. There are many other advantages of water softening, such as water free from iron and better water for heating plants, automobile radiators, cooking and making tea. There seems to be only one answer to the question, does water softening pay? The answer is yes.

Mr. Austin, the chief engineer, says the cost of softening will be completely off-set by the savings in hydrant flushing the additional industrial consumption, and the better water for the boiler plant of the municipal light plant.

The Woodstock water rates are reasonable and have not been raised on account of softening. The rates are as follows:

Minimum bill $1.50 per quarter for 5000 gallons. Next 40,000 gallons, 24 cents per M. Gal. Over 45,000 gallons, 18 cents per 1000 gal. Customers using over 500,000 gallons per quarter are billed at 13 cents per 1000 gallons. No discount for prompt payment and no penalty added for delinquent payment.

Annual expenses before operating softening plant

Annual income from sales:		
To customers	$19,570.35	
To city	6,885.00	
Miscellaneous	12.00	
Total sales		$26,467.35
Annual expenses:		
Salaries	2,200.17	
Maintenance	3,577.78	
Electric current	8,603.94	
Depreciation	2,792.76	
Total expense		$17,174.65
Net annual income		$9,292.60

CONSTRUCTION COST

The total amount of the Federal Public Works Loan and Grant was $64,000. There was about $2000 left over of this $64,000.

The construction work was divided into three contracts. The final estimates on the various contracts in round figures were as follows:

Contract 1. Structural	$24,000.00
Contract 2. Equipment	18,000.00
Total softening and iron removal plant	$42,000.00
Contract 3. Cleaning water pipe	16,000.00
Grand total construction cost	$58,000.00

The cost of cleaning water mains was 19 cents per foot. This included all labor, material and equipment.

PERSONNEL

The firm of Campbell-Lourie Lautermilch Corporation, 400 West Madison Street, Chicago were contractors on Contract No. 1, the structural work; E. W. Bacharach Company, 1200 Rialto Building, Kansas City, Missouri, were contractors on Contract No. 2, the

equipment; and the National Water Main Cleaning Company, 50 Church Street, New York City were the contractors on Contract No. 3, cleaning water mains.

Consoer, Townsend & Quinlan were the engineers for the City, and had charge of both the design and supervision of construction. Mr. Wm. F. Martin was the Resident Engineer Inspector for the PWA.

Mr. Wm. R. Burns is Mayor of Woodstock and Mr. C. K. Austin is Chief Engineer of the Water and Light Plant. The Contractors and Engineers appreciated very much the coöperation of both Messrs. Burns and Austin in carrying out the project.

(*Presented before the Illinois Section meeting, April 10, 1935.*)

THE ART OF PLANNING AS RELATED TO WATERSHED CONTROL

By M. W. Cowles

(Health Officer, Hackensack Water Company, New Milford, N. J.)

From time to time, papers have been presented before the water works associations on the subject of watershed sanitation. These usually deal with the state laws or rules and regulations relating to abatement of any existing violations, together with descriptions of problems of control over recreational activities including fishing and boating in the immediate vicinity of and in impounding reservoirs, lakes and other bodies of water from which the supply is obtained. Some water purveyors, but relatively few in number, have established a fairly comprehensive watershed program as a protection for the supply. Those in charge of watersheds have not begun, for the most part, to realize the possibilities of the word "control" or yet of "prevention." During the past 10 years, prevention through studies of causation of disease has become an integral part of the program of public health administration and has accomplished splendid results in the control of many communicable diseases. Considerable research has been done by the writer and this paper has been prepared to outline possible methods and means for developing and carrying out a comprehensive program of watershed protection over a long period of years, a program which is applicable to all water resources whether used as a source of potable water or otherwise. Such a program is therefore a part of any scheme of coördinated development either on a community or a broader basis.

The basic elements of watershed sanitation, have changed greatly in the past 20 years, due to a great variety of factors. Inaccessibility, as a protection to impounding reservoirs has largely disappeared and the present means of quick transportation over long distances has tended to increase accidental or promiscuous pollution of small tributary streams, or even of the impounding reservoir itself. Increased leisure time has become an important factor. While large scale property ownership in itself may not prevent trespass and ac-

cidental contamination it does constitute an important protective control, when accompanied by proper sanitary patrol. Marginal property ownership along impounding reservoirs including minor tributary streams and along main tributary streams a short distance from the reservoir has become necessary for the reasons given above to prevent improper uses of this adjacent land, but ownership of the greater portion of a watershed does not, at present, appear to be a sound economic principle. Marginal ownership, the extent to be determined by local conditions, should be supplemented by filtration as an absolute protection. Watershed protection is a broad public problem and all should coöperate with the water purveyor.

POLLUTION OF STREAMS

Throughout the United States there is ample evidence that, despite the continued activities of numerous governmental and semi-governmental agencies, the pollution of streams is still going on and that all these efforts have been able only to check partially the increasing rate of pollution. Much of this is due to uncoördinated activity. Marked depreciation of property has resulted where pollution has been allowed to go on to the limit unchecked. Sources of public water supply have been abandoned because of the pollution and the apparent impossibility of correcting such conditions which had grown up over a period of many years. Failure to control pollution, represents a vast economic loss which sooner or later falls upon the public treasury. Such economic losses are of a magnitude little realized. Where a stream has been allowed to become grossly polluted, for example, the Bronx River, it is inevitable that the public will be called upon to spend vast sums of money to control or eliminate the pollution. Such a condition is essentially due to the lack of recognition by public authorities of their responsibility in connection with streams as the pollution can be prevented at a minimum expense if a systematic and thorough coöperative control program is carried out over a period of years.

CHANGES IN POPULATION GROWTH

The depression has emphasized many changes which were slowly taking place in this country. One of the most far-reaching in its effect is that rate of increase of population has changed from an upward to a downward trend and the curve of actual population is flattening out. This is true of the country as a whole and also for

most communities with a few exceptions where local conditions tend to provide some further stimulation. Among other trends there are distinct tendencies for people to move from the great centers of populations to the open environs and there create a suburban type of development, rather than the distinctly rural type. All modern conveniences are now available in one form or another. Houses are on one-half to acre plots affording play space for children, some room for a garden and perhaps a few fruit trees. Lower taxes frequently add to the urge to get into the open spaces away from congestion, but the head of the house may still secure the major part of his livelihood in the great centers of population.

Studies of congestion of population in our great cities and in small communities have indicated many evils which have arisen out of our haphazard development of our communities, caused largely by the lack of foresight as the village grew into a city. Congestion of population brings with it a vast series of social and economic problems which are difficult and expensive if not nearly impossible to solve and correct. There is no more reason to permit our communities to continue to grow in a haphazard manner than there is to attempt the construction of a large building with its complicated equipment without the previous preparation of detailed plans. This is an accepted procedure and it is certain that the various parts of such a building may not function in relation to other parts and that the whole undertaking may be more or less of a failure. The construction of a water distribution system without proper design is another example. The amount of money wasted in such an unplanned undertaking is, however, a trifle compared with the total economic losses to individuals and the community as a whole which result from an unplanned community. "It is not the cost of planning that is important, but it is rather the cost of not planning," as the late Dwight Morrow so wisely said.

PLANNING MUNICIPALITIES

The art of planning of our municipal development has been gaining momentum rapidly in recent years and it is reported that there are now 763 local planning boards in the United States all operating under state enabling acts. As a result of pressure from Washington in connection with state work relief projects of all types, planning boards have been organized in 44 states in an attempt to make a study of the larger regional problems which local planning boards obviously can-

not undertake. The programs vary in several states, but, in general, are intended to cover social trends, classification of land uses, forestry, recreation, water resources, transportation and in some cases, a study of the functions of the various governmental units. Their work is being carried on slowly due to lack of funds or probably as frequently to the lack of appreciation of the real merits of the vastly complicated problem which they are attempting to solve. The New York State Board has already issued two reports which are of considerable interest.

Local planning boards created under what is known as state enabling acts are official bodies whose duty it is to study the area under their jurisdiction, to be able to devise an intelligent scheme of future development in accordance with the estimated future needs of the area. Studies of population growth of the community and neighboring communities are necessary. Studies may indicate clearly the necessity of widening of certain streets, the construction of new traffic arteries, separation of direct traffic in or around local business areas which will permit free and easy access of local people to the local shopping center and possibly grade crossing elimination. The study will indicate the location of future public buildings such as schools, municipal buildings including fire and police stations and may clearly show the desirability of ultimate abandonment of buildings which are improperly located. The provision may be made for a civic center about which a group of public buildings will later be constructed. The plan will give consideration to recreational areas needed for the existing and for the future population in the form of small neighborhood units and larger park areas. Parkways which are narrow ribbon parks, frequently containing a boulevard, may be provided along watercourses and may include certain other areas where natural topography makes development difficult yet which are ideally adopted for such purposes. In other words, the plan should provide a fairly definite pattern of development with the details subsequently to be filled in, all with due regard to all of the social economic and cultural characteristics of the community. Little by little, the community will assume the pattern desired if the plan has been wisely made. The official plan when legally adopted becomes binding on the municipality, but with a degree of flexibility to prevent unnecessary hardships. Active citizen support of a plan is necessary so that any improvement undertaken by the governing body from time to time will be consistent with the master plan.

Planning, therefore, becomes an efficient and effective guiding agent for the growth and development of the community. Planning for sanitation is too often not given sufficient consideration and should be very definitely included as a part of the program.

By far the most important duty of a local planning board is to prepare rules relating to the subdivision of vacant land into building lots and requiring approval of all plans before lots can be sold and deeds recorded. If the planning board does not act within a reasonable time, this authority usually reverts to the governing body. A wisely drawn set of subdivision regulations will provide that the land to be subdivided is suitable for building purposes and may restrict development in any particular area where soil conditions or topography are adverse. The unsuitable type of land, the most expensive to develop, is generally rather easily adopted for recreational uses. Very frequently the preliminary discussions with the developer will produce an offer to donate such unsuitable land to the municipality, especially when it is realized that the return to the developer will be considerably increased by reason of increment in value of plots which front on such an open space. New streets should be laid out in accordance with local topography. All streets need not be designed for through but only for local traffic. Provision is made for systematic installation of all utilities. If water supply and sewerage are not to be made available immediately, provision is made so that they can be installed when required. There is a very direct relation between the permanent attractiveness of an intelligently laid out subdivision and financial security of mortgages and financial investment as recently emphasized by the bulletins and procedure established by the Federal Housing Administration for insured mortgages. Adequate sanitation is likewise emphasized. The influence of the planning board in guiding the future development of the community is very far-reaching and requires the best judgment of the most farseeing citizens of the community.

CONTROL OF REAL ESTATE DEVELOPMENT

To show the disastrous results introduced by our present system of control of building construction in sparsely settled areas, it is only necessary to refer to a type of development, one of which is doubtless in your own community, where perhaps in 1920 or thereabouts, a real estate developer sold a tract of land by cutting it into 25 foot lots, selling one lot to a purchaser, at a very low price. No thought

was given to water supply or sewage disposal. No attempt was made to fit the development to local topography. Some lots cannot have access except over streets with excessive grades and the expense of development prevents their use. The municipality may have approved the plans, the idea of development being sold to the community on a high pressure basis. Scattered small houses were erected. Sanitary conditions became very bad. Private wells became polluted and sewage was discharged into the streets. Many of the purchasers have been unable to pay their taxes or to keep up their payments and scattering lots have reverted to the community. The developer has made his money and left for parts unknown leaving to the community a real problem. The assessed valuation of all the properties ten or fifteen years later is insufficient to enable the installation of an adequate public water supply or a sewerage system for the topography has added greatly to the expense. The character of population has deteriorated and the whole area has become little short of a country slum involving many of the problems which are connected with slums in large cities. Probably nothing short of actual purchase or condemnation by the municipality directly or through some housing authority can correct the conditions in such an area. A study of the actual cost of such an inferior development to the municipality and its citizens would doubtless yield some very interesting comparisons.

Planning boards having authority to consider all phases of development are generally quick to realize that larger lots with a frontage of 50 to 75 or even 100 feet and a depth of 125 to 150 feet, represent a much greater asset to the community than do 20 or 25 foot lots with a depth of 75 to 100 feet. Even where the smaller lots are sold three to a purchaser the owner of a single lot will appear from time to time and wish to build thereon. Where public water supply and sewerage facilities are not immediately available, the larger lots will permit the installation of a private sewage disposal device such as septic tank and drain tile and also allow for a safe source of water supply all on the same lot, providing of course the locations of similar facilities on adjacent lots are properly worked out. Where dwellings are erected on the larger lots, they do not constitute a menace to the health of the community or to the health of the individuals living therein, nor does the development involve any deterioration of the quality of water flowing in any of the watercourses.

Recent tendencies in real estate development in the country have

been to emphasize attractiveness and the value of watercourses in connection with community development. The land immediately adjoining the stream is generally of low value compared with other land in the community. Where a strip of open land is left along a watercourse, the value of the stream can be translated into greater property values of the abutting land. The actual assessed valuation in such a section of the community can definitely be shown to be greater than if development were allowed to take place immediately adjacent to the stream. In this way, streams are not only protected against contamination, but are made to serve a useful purpose and are of lasting benefit to the community as a whole.

Lakes, ponds and the larger streams have, with the advent of the automobile, become very attractive for summer vacation areas. It is most important that such developments be controlled and the serious need of systematic planning is emphasized, otherwise, these places lose much of their attractiveness and their value. In general, the communities in which these recreational areas are located are not awake to the situation and some simple form of control by some state agencies appears to be very necessary.

One is astonished at the rapid extension of the idea of municipal zoning which is based upon the exercise of community power (so-called police power) which through state enabling acts authorize the local governing bodies to exercise control over new buildings. This movement started in New York City in 1916. Ample provision is made for light, air and open space and the concentration of population can be definitely controlled and limited. The municipality is divided into use districts where for example one-family houses, apartments, business and industry are permitted. Zoning is not intended to provide an arbitrary limitation on building, but rather is intended to be a harness which will control growth. It is capable of flexibility not only to respond to growth, as for example an increase in the area zoned for business, but also to prevent unnecessary hardships in individual cases. It prevents considerable deterioration of property values, but the maximum possibilities are as yet unrealized.

The zoning commission in the exercise of its authority over open spaces around buildings can make requirements of ample yard area for every building to provide for private sewage disposal devices and often for safe private water supply. The open space required for such facilities is in general not in excess of what prudent judgment of a person trained in city planning would feel necessary for the best

interest of the community. Public water supply and sewers may come later, but they are not vitally needed to control sanitary conditions. A wisely drawn zoning ordinance is, therefore, capable of providing a certain type of control to supplement the activities of the planning board and enforcement of the local building code.

CONTROL OF WATERSHEDS

Nearly all of the larger watersheds in this country from which a potable water supply is drawn, have built-up communities upon them of varying size some of which are rural villages developing very slowly and others thriving, growing towns. For the latter type where there is the most danger of contamination of watercourses through uncontrolled growth, the creation of planning boards and the adoption of zoning ordinances can be fairly readily accomplished. In the rural areas it is much more difficult to enlist interest in these matters, but here some simplified or skeleton control with simple administration can doubtless be worked out either locally or without outside assistance.

The burden of the protection and prevention of objectionable conditions is directly a public health problem and should not fall entirely on the water purveyor. It is a matter of coördinated and coöperative action in the public interest. Where the public health phase is adequately handled and controlled the problem of watershed control is greatly simplified. The inherent strength of the suggestions made would at first thought, indicate that the necessary controls over development would be established. The writer has done considerable research on this problem of control and certain interesting developments have come to light which can be only referred to at this time. The growth of a community regardless of its location occurs rather obviously, house by house, or building by building. What other official agencies of the municipality are concerned and what part, if any, can they play in the control program? The local building inspector must issue a permit for the building, providing it is in accord with the building code which relates primarily to safety and also in accord with zoning and other ordinances, but with little or no emphasis on the all important item of sanitation as it may effect those persons residing in the building and on adjoining property and the community in general. Permits for the plumbing system within the building are issued by an official of the local board of health or by a representative of the building department in accordance with an adopted plumbing code.

Plumbing codes can be roughly divided into two general classes (1) a state plumbing code of minimum requirements supplemented and amplified by locally adopted codes not inconsistent therewith and, therefore, more or less of the same pattern and (2) local codes enacted without state supervision and guidance. The second group shows a wide range of subjects covered and an even wider range of requirements on a particular subject. Sanitary codes may be classified in a somewhat similar manner and only too frequently they relate largely, at least as far as disposal of human waste is concerned, to matters of nuisance of one kind or another, many of which are relatively unimportant to the public health in the broadest sense. Under this type of code, however, most of the disease prevention work is accomplished by local health authorities. The researches of the writer thus far made indicate that a scheme can be devised which will carry into effect the principle of "prevention" in a simple and direct manner through a properly drawn series of codes containing provisions relating to the construction of private sewage disposal devices. Such a scheme could perhaps be made to control legally and effectively the erection of new buildings and prevent their erection where the owner is unwilling or unable to provide adequate sanitary facilities.

Reference has already been made to the necessity of control of development at summer resorts along our lakes and streams. The state codes of Wisconsin adopted in 1926, contain provisions for proper sewage disposal in all locations and also provide for a rigid control over new subdivisions along its lakes and streams. The results have apparently been exceedingly successful, but the expense of supervision may be somewhat heavy. The reports of the New York State Planning Board have emphasized very strongly the necessity of conserving and protecting such areas but thus far have not made any specific suggestions. In 1933, a law was passed in New York State requiring submission of plans to the State Health Department where developments with more than 10 houses were proposed, the plans to include a topographical survey, detailed information regarding the size of lots and proposed water supply and sewerage facilities. In the opinion of the writer, this law represents a step in the right direction, but it is not broad enough to effect adequate control of the development along the lakes and streams of that state. It is evident that the state has a distinct responsibility in connection with such developments and should take steps to control them

properly. Having in mind the relation of the state to its communities, it would seem readily possible to provide a fairly ideal set-up of control, represented by state laws or codes of minimum basic requirements enforced by the local governing bodies with the assistance and support of the state wherever necessary. Planning and zoning on a simplified or skeleton basis can be developed as an essential part of any such comprehensive plan.

The writer has been secretary of an unofficial county planning association, member of two official zoning commissions and is now vice-chairman of the official Planning Board of the Village of Ridgewood, New Jersey. Since 1926, he has been in charge of watershed sanitation for the Hackensack Water Company of which Nicholas S. Hill, Jr., is President. This broad experience has given definite assurances that these several phases of watershed control have interesting future possibilities of great importance in the problem of conservation of our water resources as well as in the growth and development of our communities. If the principles of public health are carried out in an efficient and continuing manner, watershed control would be much simplified.

CONCLUSIONS

Coördinated planning activity in which state, county and local municipality share together with zoning on a somewhat similar basis can be expected to accomplish much in the way of watershed control over a long period of years through prevention, thereby resulting in conservation of our water resources. Community sanitation should be given a most important place in all planning studies.

Planning and zoning even more broadly interpreted than they have been in the past will provide a framework into which from time to time, the desirable details can be fitted so as to provide a complete and workable pattern of the greatest possible value to our citizens.

(*Presented before the New York Section meeting, March 22, 1935.*)

STERILIZING VELOCITIES OF CHLORINE AND CHLORAMINE UNDER VARYING CONDITIONS OF ORGANIC LOAD AND pH[1]

By Paul J. Beard[2] and Nathaniel J. Kendall[3]

(From the Laboratories of Sanitary Engineering and Bacteriology and Experimental Pathology, Stanford University, Calif.)

The investigations of Holverda (1) and of Baker and Schmelkes (2) indicate that the variations in data on the sterilizing velocity of chloramines are, in part at least, due to variations in hydrogen ion concentration. These experiments and those of others, Gerstein (3) and Hinman and Beeson (4), were with waters carrying none or negligible amounts of organic matter. Gerstein carried out experiments on Lake Michigan water, but does not mention its organic content or chlorine demand. Hinman and Beeson worked with waters of known but low organic load.

That hydrogen ion concentration is intimately related to the velocity of sterilization is well recognized, but the effect on this velocity of variations in organic load have not been adequately investigated.

The object of the present studies were:

1. To compare the velocity of chloramine sterilization of water at various hydrogen ion concentrations.

2. To compare these velocities in waters of varying organic load.

The studies were entirely on a laboratory scale, each *run* consisting of a 250 cm.[3] volume containing the organic matter, test organisms and sterilizing agent. Hinman and Beeson's[4] procedure of ammonia and chlorine application were followed.

The chlorine solution was made by bubbling chlorine gas through

[1] Extract of thesis presented to Committee on Graduate Study in partial fulfillment of requirements for the Degree of Engineer.

[2] Assistant Professor Sanitary Sciences.

[3] Graduate Student.

[4] Loc. cit.

We acknowledge the courtesy of the Wallace Tiernan Co. in supplying the chlorine used in these studies.

distilled water until a concentrated chlorine water was obtained. The strength of the solution was determined by titration with sodium thio-sulphate. The methods used were those outlined in *Standard Methods of Water Analysis*. Sufficient dilute amounts of this chlorine water were used in the tests to give the required chlorine content.

The ammonia water used was a commercial solution about a year old. Its ammonia content was determined by direct nesslerization and by calculation from the label data. This solution was found to be quite stable.

In the determination it was necessary to buffer the solutions to maintain the desired hydrogen ion concentrations. The buffer solutions were prepared as outlined by Kolthoff and Furnam. Mixtures of primary phosphate (KH_2PO_4) 1.0 mol. and borax ($Na_2B_4O_7$) 0.5 mol. solution, were used in the preparation of buffer mixtures. For a pH of 6.6, 7.22 cc. of KH_2PO_4 and 2.78 cc. of borax was used; for a pH of 7.4, 5.50 cc. of KH_2PO_4 and 4.50 cc. of borax was used; and for a pH of 8.2, 4.30 cc. of KH_2PO_4 and 5.70 cc. of borax was used.

ORTHO-TOLIDINE DETERMINATION OF FREE CHLORINE

The ortho-tolidine method of the determination of free chlorine was used. This method was supplemented by the use of a Wallace and Tiernan colorimeter. Good results were obtained until tap water was used. Then a blue color developed upon addition of the o-tolidine reagent.

Ellms and Hauser (5) state that waters which are naturally alkaline from the carbonation of calcium and magnesium, which they contain, or those rendered artificially so by any of the fixed alkalines, set similarly. The higher the original alkalinity of the water containing free chlorine, the bluer is the shade of color produced. This they state takes place in the solution when the ortho-tolidine reagent has been made up in acetic acid solution. They suggest the addition of a small quantity of a highly dissociated acid such as hydrochloric or sulphuric. This produces a deep yellow color with small amounts of free chlorine and the color is not affected by the original alkalinity of the water tested nor is the tint changed by any concentration of the acid. The blue color obtained during the tests was changed to the characteristic yellow by the addition of a few drops of concentrated hydrochloric acid.

They also state that if the ortho-tolidine reagent is made up in a hydrochloric acid solution, a uniform yellow color is produced with

small amounts of free chlorine regardless of the soluble constituents of the water tested. They assert that the presence of sulphates, chlorides and nitrates of alkalies and alkaline earth bases do not interfere with the test, that the yellow color develops in about three minutes and lasts a half hour. *Standard Methods* suggests the addition of more acid when the tints do not match the tints of the standards as may be the case in certain lime treated or natural waters. *Standard Methods* uses the HCl solution of o-tolidine as an indicator.

The blue color is produced, according to Ellms and Hauser, as a result of the oxidation of the amino-group, and not the methyl group, since in aniline and benzidine, having no methyl groups, similar dyes are formed by the action of hypochlorites. Leech (6) suggests that the blue color reaction is produced by the action of hypochlorites on methyl-aniline and ethyl aniline. Also that the blue dye, possibly a member of the "indo" series of dyes, results from the oxidation of the methyl or ethyl group.

Race (7) obtained good results except for colored waters, in which the coloring matter diminishes in intensity on the addition of acids and is somewhat similar in tint to that produced by ortho-tolidine. If the reaction is used qualitatively on colored treated water and a comparison made with the untreated sample, a negative result might be obtained where traces of free chlorine are present, because the reduction in color produced by the acid is greater than the increase caused by the reagent. Similar difficulties would also be encountered when quantitative results were attempted.

Adams and Buswell (8) in their work on the ortho-tolidine test for chlorine found that similar color reactions were produced with manganic compounds as with free chlorine, thus proving quantitatively that the color formation was an oxidation reaction rather than one of chlorination and that chloramines give a similar ortho-tolidine reaction as equivalent amounts of chlorine, but that a longer time is necessary for full color development.

Buswell and Boruff (9) found that the o-tolidine method gave consistent results between a pH of 3.0 and of 8.0, that low temperature lengthened the time for maximum color and gave a less sensitive reaction, and that the oxidized organic compounds had no effect on the test.

The author checked the results of the ortho-tolidine test by the starch-iodide indicator and titration with sodium thiosulphate at various intervals. Very consistent results were obtained by the

two methods. A Wallace and Tiernan colorimeter was also used during most of the determinations as a check and comparison to the ortho-tolidine test.

The experimental work that will be discussed in this paper has been derived from studies at three hydrogen ion concentrations, pH 8.2, 7.4, and 6.6, which probably include the extremes commonly encountered in waterworks practice. Varying organic loads from 0 to 300 p.p.m. under these conditions of hydrogen ion concentration, likewise representing probable extremes, have been studied. For the purpose of convenience these data have been tabulated and graphically represented in a series of charts. The discussion of the individual experiments and their significance follows.

DISCUSSION OF RESULTS

The data developed from this work must be discussed, not only as individual experiments, but also in groups according to the various controlling constants and variables. These factors are hydrogen ion concentration (pH), organic content, contact period, the amounts of chlorine and ammonia used, and the chlorine residuals.

The relative velocities of chloramine sterilization at the different hydrogen ion concentrations (pH) are well shown by figure 1. Here time is plotted against the percent removal, the organic matter being constant at zero. Three pH values (8.2, 7.4, and 6.6) were investigated. At pH 6.6 the velocity of sterilization was very rapid, giving almost maximum sterilization in 5 to 10 minutes. Whereas at pH 7.4, the velocity was much slower and 45 minutes were necessary before good sterilization took place. Then the curve begins to straighten out. At a pH of 8.2 the velocity of sterilization was quite gradual, almost rising as a straight line, giving only poor sterilization in 60 minutes. The runs plotted were those in which only a slight residual was carried. This was done so that the velocity would be slower and more accurate data obtained.

The results of the runs in which a 0.2 to 0.3 p.p.m. chlorine residual was carried showed very rapid sterilization for the two lower pH values. But at pH 8.2 a decided lag for 10 to 15 minutes was observed.

Upon the addition of increasing amounts of organic matter (peptone), the lag at pH 8.2 becomes progressively quite noticeable. At high organic loads (300 parts per million), the velocity of steriliza-

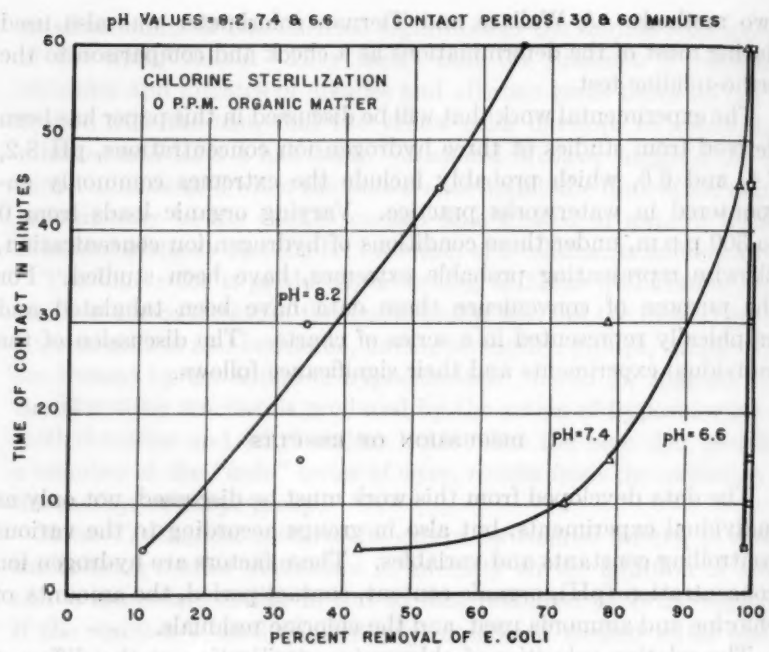
pH VALUES = 8.2, 7.4 & 6.6
CONTACT PERIODS = 30 & 60 MINUTES
CHLORINE STERILIZATION
0 P.P.M. ORGANIC MATTER
TIME OF CONTACT IN MINUTES
pH = 8.2
pH = 7.4
pH = 6.6
PERCENT REMOVAL OF E. COLI

FIG. 1

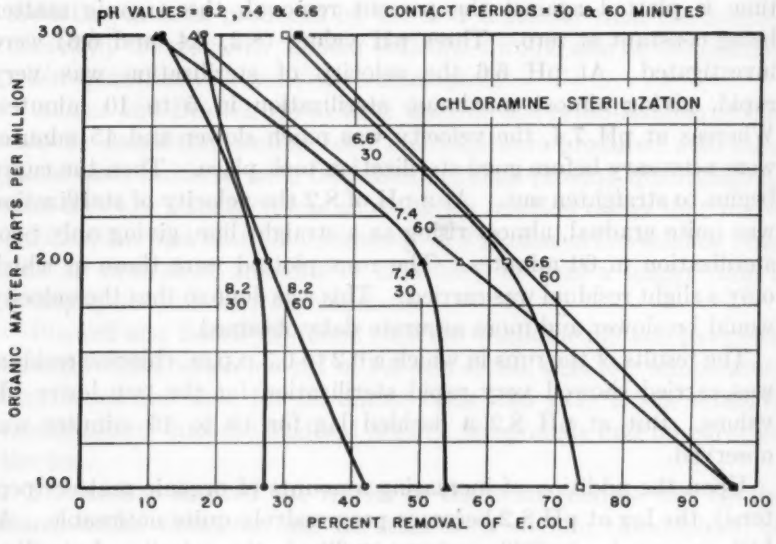
pH VALUES = 8.2, 7.4 & 6.6
CONTACT PERIODS = 30 & 60 MINUTES
CHLORAMINE STERILIZATION
ORGANIC MATTER - PARTS PER MILLION
6.6 30
7.4 60
6.6 60
7.4 30
8.2 30
8.2 60
PERCENT REMOVAL OF E. COLI

FIG. 2

tion at pH 8.2 and 7.4 are about the same; whereas the velocity of sterilization at pH 6.6 is much higher.

From figure 2 it can be seen that the velocity of sterilization slows down considerably with higher organic loads. This is more noticeable at the lower hydrogen ion concentrations, in which the curves show that there is a more gradual decrease in the velocity at pH 8.2, whereas at pH 7.4 and 6.6 the change is much more rapid. This chart also shows a tendency toward a lag at the beginning of sterilization. This is not as evident at pH 8.2 as the lag there seems to extend for quite a long period. But at pH 7.4 and 6.6, the curves for 30 minute sterilization are not particularly different, in that of organic loads of 100 and 200 p.p.m., the percentage removal is not dissimilar to a great degree in both cases. But for a 60 minute contact period, a great difference in the velocity of sterilization results from increased organic loads. That is, for an increase in organic matter there is a decided drop in the percent removal of bacteria.

The data plotted on figure 2 were obtained from runs in which the chlorine and ammonia added were kept constant. Thus at the higher organic loads a greater chlorine demand was built up, the velocity of sterilization retarded, and the completeness of removal diminished. Three hundred parts per million organic matter, the upper limit in these experiments, were used as a limit, because it is probably a maximum that might be found in a water, as it is comparable to the organic load of a weak residential sewage.

With an increase in the organic load, therefore, giving a resulting increase in chlorine demand, the velocity of sterilization will be repressed. But with the organic load constant, greater and faster sterilization results at the lower pH ranges. This might be explained by the relative amounts of the two chloramines present in the solution. Race (7) has stated that the activity of the chloramines might be comparable to their heats of formation. The heat of formation of mono-chloramine is 8,230 calories, of di-chloramine 36,700 calories and of tri-chloramine 65,330 calories. Thus from the above it might appear that tri-chloramine would be the most active, but it would be impracticable to use it in water purification because of the very low pH necessary for its formation. But from the data as above, it is evident that the dichloramine is the most active (of the two used in water works practice,—mono- and di-), if their activity as a sterilizing agent is comparable or proportional to their heats of formation.

This is borne out by the experiments. For example, at a pH of

6.6 where there is an excess of di-chloramine over mono-chloramine (63 percent di-chloramine and 37 percent mono-chlorine according to Chapin's chart) the velocity and amount of sterilization is much greater than at the higher pH values where mono-chloramine predominates.

At pH 7.4 where about 65 percent of the chloramine mixture is mono-chloramine and 35 percent di-chloramine, the velocity and amount of sterilization is not as great as at the lower pH. This fact is emphasized by the experimental data at pH 8.2 where the di-chloramine content is only about 8 percent of all the chloramine mixture present.

TABLE 1

Chloramine and chlorine sterilization

A natural water, 150 p.p.m. organic matter

pH	TIME	CHLORAMINE		CHLORINE RESIDUAL	PERCENT REMOVAL E. COLI
		Cl_2	NH_3		
	minutes	*p.p.m.*	*p.p.m.*	*p.p.m.*	*percent*
8.2	30	1.8	1.2	0.05	42.2
8.2	60	1.8	1.2	0.05	52.5
7.4	30	1.8	1.2	0.04	61.2
7.4	60	1.8	1.2	0.04	80.0
6.6	30	1.8	1.2	0.04	74.6
6.6	60	1.8	1.2	Trace	97.8

Thus one may conclude that the greater the concentration of di-chloramine in the treated water the greater the velocity and amount of sterilization that will take place. This is not in agreement with previous observers. According to the available data in the literature the greatest chlorine efficiency has been stated to be on the alkaline side of the reaction. This has probably been a result of Chapin's (10) statement that mono-chloramine is the better germicidal agent.

Since all of the previous work had been secured with artificially "loaded" water, it was desirable to compare these data with those to be secured on a water containing a fairly high natural organic content. To this end a lake water with appreciable turbidity and color (80, La Motte standards) was investigated. The organic matter as determined by loss of weight on combustion was 150 p.p.m.

The results were altogether in harmony with what might have been expected from the work with peptone water. The results of this experiment are shown in table 1. These data, as can be seen, are quite comparable with those on figure 2. It is quite significant that the data show the best removal of E. coli at a pH of 6.6 and a relative decrease at the two higher hydrogen ion concentrations.

The relative velocities of chlorine sterilization are clearly shown in figure 3. Here again in straight chlorination the best results were

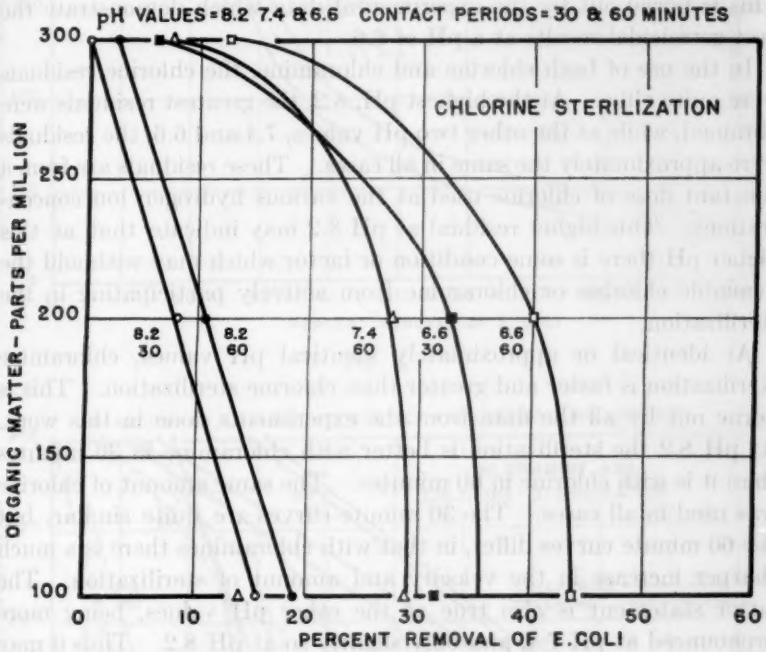

FIG. 3

obtained at the lower pH concentrations. Although these data did not develop as smooth and well defined curves as in the case of chloramine sterilization, the general trend is the same. In the data at pH 8.2 and 6.6 the 30 and 60 minute contact curves are quite parallel, and show the greater sterilization at the longer periods. From these curves one may deduce that, with a very low organic load, there would not be significant differences between the velocities of sterilization at the various pH values. From the runs made with a more

organic load and chlorine sterilization, it can be seen that there was not a great difference, although it was best at the lower pH values.

With the addition of organic matter the velocity of sterilization was somewhat decreased, but the change was not great until 300 p.p.m. were added. Then the drop was quite noticeable. The results of the straight chlorine experiments do not show a decided lag in the velocity of sterilization. According to Horwood the velocity of the germicidal action of chlorine in water is increased perceptibly in an acid solution, whereas an alkalinity tends to reduce it. This is borne out by the experimental data which demonstrate the best germicidal results at a pH of 6.6.

In the use of both chlorine and chloramine, the chlorine residuals were quite alike. At the highest pH, 8.2, the greatest residuals were obtained, while at the other two pH values, 7.4 and 6.6, the residuals were approximately the same in all cases. These residuals are from a constant dose of chlorine used at the various hydrogen ion concentrations. This higher residual at pH 8.2 may indicate that at this higher pH there is some condition or factor which may withhold the available chlorine or chloramine from actively participating in the sterilization.

At identical or approximately identical pH values, chloramine sterilization is faster and greater than chlorine sterilization. This is borne out by all the data from the experiments done in this work. At pH 8.2 the sterilization is better with chloramine in 30 minutes than it is with chlorine in 60 minutes. The same amount of chlorine was used in all cases. The 30 minute curves are quite similar, but the 60 minute curves differ, in that with chloramines there is a much sharper increase in the velocity and amount of sterilization. The latter statement is also true at the other pH values, being more pronounced at pH 7.4, and only slightly so at pH 8.2. Thus it may seem that chloramine sterilization is far superior to chlorine sterilization in velocity and amount.

Organic matter appears to exert a relatively greater effect upon chloramine than upon chlorine efficiency. This is true at the higher organic loads and can be demonstrated by the data on figure 5, where a greater variation in the velocity of chloramine sterilization occurs at the higher organic loads than with chlorination.

From figure 4 it can be seen that at pH 7.4 the velocity and amount of chlorine sterilization falls off quite rapidly between 0 and 100 p.p.m. of organic matter. Whereas the chloramine velocity and

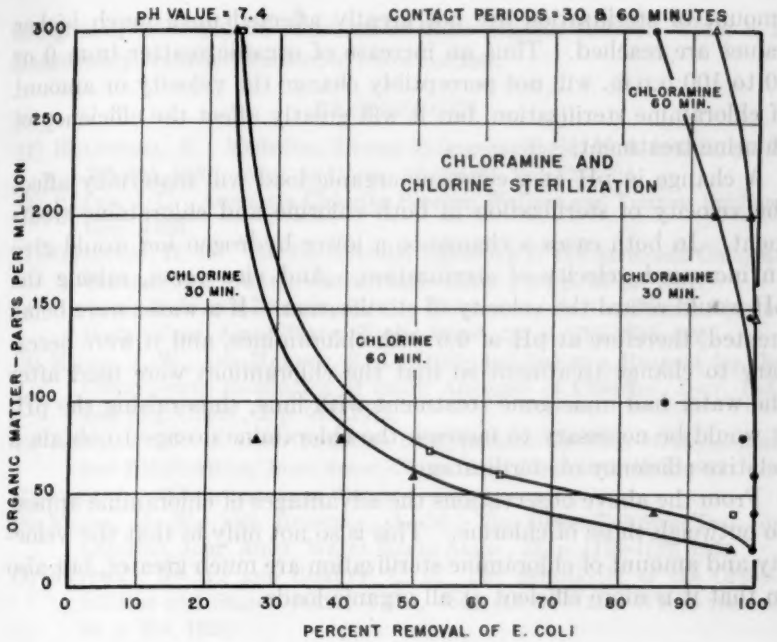
pH VALUE = 7.4
CONTACT PERIODS = 30 & 60 MINUTES
CHLORAMINE AND
CHLORINE STERILIZATION
CHLORAMINE
60 MIN.
CHLORINE
30 MIN.
CHLORINE
60 MIN.
CHLORAMINE
30 MIN.
ORGANIC MATTER - PARTS PER MILLION
PERCENT REMOVAL OF E. COLI
300
250
200
150
100
50
0
0
10
20
30
40
50
60
70
80
90
100

FIG. 4

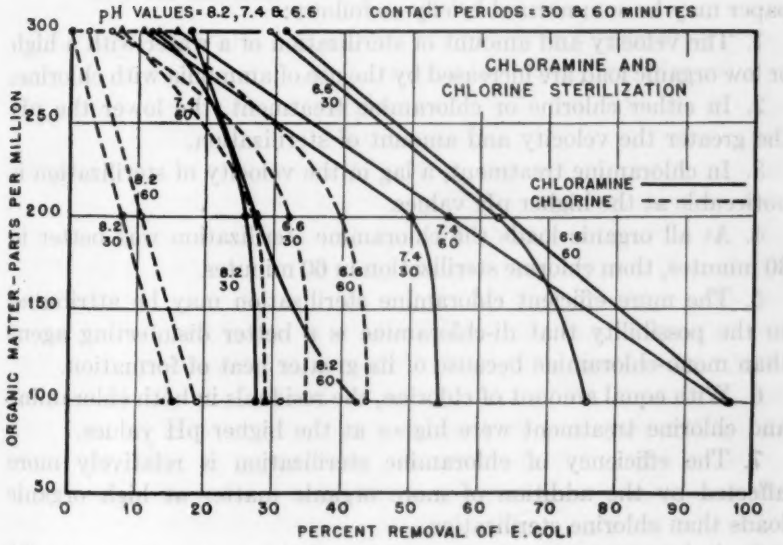
pH VALUES = 8.2, 7.4 & 6.6
CONTACT PERIODS = 30 & 60 MINUTES
CHLORAMINE AND
CHLORINE STERILIZATION
CHLORAMINE
CHLORINE
6.6
30
7.4
60
8.2
60
8.2
30
6.6
30
8.2
30
6.6
60
7.4
60
7.4
30
6.6
60
8.2
60
ORGANIC MATTER - PARTS PER MILLION
PERCENT REMOVAL OF E. COLI
300
250
200
150
100
50
0
10
20
30
40
50
60
70
80
90
100

FIG. 5

amount of sterilization are not greatly affected until much higher values are reached. Thus an increase of organic matter from 0 or 10 to 100 p.p.m. will not perceptibly change the velocity or amount of chloramine sterilization, but it will greatly affect the efficiency of chlorine treatment.

A change in pH at a constant organic load will materially affect the velocity of sterilization in both chlorine and chloramine treatment. In both cases a change to a lower hydrogen ion would give an increased velocity of sterilization. And vice versa, raising the pH would retard the velocity of sterilization. If a water were being treated, therefore at pH of 6.6 with chloramines, and it were necessary to change treatment so that the chloramines were used after the water had undergone treatment with lime, thus raising the pH, it would be necessary to increase the chloramine dosage to obtain a relative efficiency of sterilization.

From the above observations the advantages of chloramine appear to outweigh those of chlorine. This is so not only in that the velocity and amount of chloramine sterilization are much greater, but also in that it is more efficient at all organic loads.

CONCLUSIONS

The conclusions drawn from the experiments described in this paper may be summarized briefly as follows:

1. The velocity and amount of sterilization of a water with a high or low organic load are increased by the use of ammonia with chlorine.
2. In either chlorine or chloramine treatment, the lower the pH the greater the velocity and amount of sterilization.
3. In chloramine treatment, a lag in the velocity of sterilization is noticeable at the higher pH values.
4. At all organic loads the chloramine sterilization was better in 30 minutes, than chlorine sterilization in 60 minutes.
5. The more efficient chloramine sterilization may be attributed to the possibility that di-chloramine is a better disinfecting agent than mono-chloramine because of its greater heat of formation.
6. With equal amount of chlorine, the residuals in both chloramine and chlorine treatment were higher at the higher pH values.
7. The efficiency of chloramine sterilization is relatively more affected by the addition of more organic matter at high organic loads than chlorine sterilization.
8. At low organic loads the velocity of chloramine sterilization is

not greatly changed on addition of more organic matter, while it is measurably retarded in simple chlorination.

REFERENCES

(1) HOLWERDA, K. Mededeel. Dienst Volksgezondheid, Ned. Ind. 19, pt. 2, 325–365, 1930. Abst. in Chem. Abstracts, Vol. 23, 1704.

(2) BAKER, J. C. Ammonia Chlorine Process, Wallace and Tiernan Publication #305.

(3) GERSTEIN, H. H. The Bactericidal Efficiency of the Ammonia Chlorine Process, Jour. Amer. Water Works Assoc., 23, p. 53–62, 1931.

(4) HINMAN, J. J. AND BEESON, K. C. Chloramines in the Disinfection of Water, Jour. Amer. Water Works Assoc., 21, p. 1705–1716, 1929.

(5) ELLMS, J. W., AND HAUSER, S. J. Ortho-tolidine as a Reagent for the Colorimetric Estimation of Small Amounts of Chlorine, Jour. Ind. and Eng. Chem., 5, p. 915, 1913.

(6) LEECH, P. M. A Color Reaction of Hypochlorites With Methylaniline and Ethylaniline, Jour. Amer. Chem. Soc., 35, p. 1042–1044, 1913.

(7) RACE, J. "Chlorination of Water," 1918.

(8) ADAMS, H. W., AND BUSWELL, A. M. The Ortho-Tolidine Test for Chlorine, Jour. Amer. Water Works Assoc., 25, p. 1118–1139, 1933.

(9) BUSWELL, A. M., AND BORUFF, C. S. The Sensitivity of the Ortho-tolidine and Starch Iodide Tests, Jour. Amer. Water Works Assoc., 14, p. 384, 1925.

(10) CHAPIN, R. M. The Influence of pH Upon the Function and Decomposition of Chloro Derivatives of Ammonia, Jour. Amer. Chem. Soc., 53, p. 912–920, 1931.

EVALUATION OF RESIDUAL CHLORINE

By A. E. Griffin

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Ortho-tolidine has been used almost exclusively for the estimation of residual chlorine in water since its standardization in 1913 by Ellms and Houser (1), and its development by Wolman and Enslow in 1918 (2). Prior to that time the then less delicate starch iodine method had been used. In addition to its extreme sensitivity it has the advantage of requiring but one manipulation, namely, the addition of the indicator, which produces a color varying in intensity with the quantity of chlorine present. It was early recognized that this new reagent reacted to certain materials, other than chlorine as is attested by the warning in the 1917 issue of "*Standard Methods*" that nitrites, ferric salts and chlorates react with ortho-tolidine. Unfortunately, however, no method of correction for these substances was given. Even the 1933 edition of "*Standard Methods*" does not suggest methods of correction for various interfering substances other than by reference to several articles.

Between 1913 and 1935 cumulative experience has given us a fairly complete picture of what to expect of this compound. For instance, we know that the yellow color is produced by oxidation (3) in an acid solution. Therefore, strong oxidizing agents such as chlorine, bromine, manganese and ferric compounds will turn it yellow. We also know that the reaction of nitrites with ortho-tolidine produces a yellow color due to diazotization. However, it is not so generally known that lignin in ground wood pulp will also react with ortho-tolidine to produce a yellow color which will remain stable even in bright sunlight over a period of days. In fact, dimethylparaphenylenediamine (4) an organic material somewhat similar in composition to ortho-tolidine also produces a red color with lignin in wood and is used by paper makers for estimating its percentage in paper. O.T. could be used in a similar manner except that the color produced is rather weak.

The accumulating difficulties through the past decade regarding the substances which interfere with ortho-tolidine have tended to throw some doubt upon the value of this really excellent indicator. Many other compounds have been investigated in an attempt to find some material sensitive to residual chlorine and at the same time unaffected by outside influences. To date none has been found. Tarvin, Todd and Buswell (5) investigated more than 50 compounds and conclude that "ortho-tolidine was found to be the best reagent for the determination of chlorine of the compounds investigated."

INTERFERING SUBSTANCES

The chief substances which give trouble in the estimation of residual chlorine with ortho-tolidine are manganese, nitrites, and iron. Organic coloring matter is mentioned in the 1922 "Standard Methods of Water Analysis" as interfering with the test, but little experimental work has been published concerning it.

Iron

We know that iron in combination with organic matter appears to increase the activity of ferric ions toward ortho-tolidine (6). Scott shows that when equal amounts of iron are added to waters containing varying amounts of organic matter the color produced with ortho-tolidine varies with the organic impurities.

Scott obtained varying apparent residual chlorine readings in waters containing different amounts of organic matter while the writer obtained still different values when using waters prevailing in Northern New Jersey. These experiments were conducted without the addition of chlorine. All of these observations indicate the hazard of attempting to apply a standard correction for iron.

These results led to experimentation with the effect of various combinations of iron, organic matter and chlorine on the ortho-tolidine test.

Three samples were used, one containing 1.0 p.p.m. Fe and 0.5 p.p.m. Cl_2; one containing 1.0 p.p.m. Fe but no chlorine, and the third containing 0.5 p.p.m. Cl_2 without iron. A reaction time of 10 minutes was allowed for the chlorine. At the end of that time ortho-tolidine was added to each sample and the color development observed at various intervals. The results obtained are shown in table 1.

These results serve to demonstrate that iron in water in moderate

amounts (less than 1.0 p.p.m.) may have little effect on the ortho-tolidine test. This is particularly true when the readings are made within 20 minutes. Since the iron content of the average domestic water seldom exceeds 0.3 p.p.m. it would appear that the true residual is affected very little thereby. However, every water works chemist should know about these reactions and apply them to the water with which he is working so that he will know exactly what he is doing.

When more than 1.0 p.p.m. was added to tap water results were obtained as shown in table 2.

TABLE 1

Comparison of effect of presence and absence of iron on chlorinated and unchlorinated unfiltered tap water

TIME OF CONTACT WITH ORTHO-TOLIDINE (MINUTES)	1.0 P.P.M. Fe 0.5 P.P.M. Cl_2	1.0 P.P.M. Fe NO Cl_2	NO IRON 0.5 P.P.M. Cl_2
5	0.16	tr.	0.19
10	0.19	0.02	0.20
15	0.20		0.20
25	0.22	0.05	0.20
45	0.25	0.12	0.20

TABLE 2

Effect of 2, 10 and 25 p.p.m. Fe on ortho-tolidine in tap water

Fe ADDED (P.P.M.)	APPARENT RESIDUAL (IN P.P.M.) AFTER MINUTES CONTACT WITH ORTHO-TOLIDINE			
	10	20	30	60
2	0.05	0.06	0.09	0.22
10	0.15	0.35	0.65	0.90
25	0.30	0.70	1.20	1.60

These results indicate when more than 1 p.p.m. Fe is present very serious errors may occur. These results probably will not be duplicated for varying amounts and perhaps even different types of organic matter will profoundly affect the results. Iron of the above order would not ordinarily be encountered. Where it is encountered in such large quantities it has been eliminated in the past by precipitating the iron with an excess of sodium hydroxide. This treatment converts the chlorine present into sodium hypochlorite which may then be accurately tested for chlorine content either by direct titrations with thiosulphate or by the usual ortho-tolidine method.

Temperature

Buswell and Boruff (7) showed in 1925 that "as the ortho-tolidine test depends on an oxidation process the depth of color varied directly as the temperature." They conclude that since little change occurs in the temperature encountered in ordinary water works practice this effect can be discounted. At Bound Brook, New Jersey, it was found that when the sample was brought to an average of 70°F. before the addition of O.T. more chlorine was recorded than when the indicator was added to the cold sample. In some instances this difference was as much as 50 percent.

Enslow and Scott and others have also called attention to this characteristic. All suggest bringing the temperature to 20°C. (68°F.) before the addition of ortho-tolidine. This seems very sensible for it would have the effect of standardizing the temperature through the year. The difference between the hot and cold residuals are greater with chloramine than with Cl_2 alone.

Manganese

Manganese is more widespread throughout the country than has been commonly appreciated. In several cases its presence has caused rather serious operating difficulties. Forman, Scott, Enslow, Hopkins, Campion, Braidech and Buswell have all proposed methods for its elimination in the test for residual chlorine. These suggestions have all helped and at least show where to start. Although they all appear to have their drawbacks, Scott (8) now gives a method whereby manganese may be removed from the solution being tested by flocculation with magnesium sulphate and sodium hydroxide. After centrifuging at 1000 r.p.m. for 3 minutes the supernatant is decanted off and tested with the modified ortho-tolidine. The reading is made after 5 minutes. It is evident that this method does get rid of interfering iron and manganese, but it also has the drawback of being a somewhat complicated procedure and thus does not make itself available for the average water supply operator.

The New York State Department of Health advocates a method for the elimination of manganese in the absence of chloramines which is practical and requires no extra apparatus as follows: Collect two samples at the regular sampling point. Determine residual on one at once allowing the other to stand for 24 hours. At the end of this time determine residual on this second sample, subtracting result from the value of the first. The difference will represent the true

residual. The accuracy of this test, of course, depends upon the dissipation of the chlorine within 24 hours.

Enslow prefers to boil off the excess chlorine subtracting the residual obtained after boiling from the cold residuals. This also has the advantage of simplicity.

With these latter three methods available one should now be able to approximate closely the actual residual in the presence of manganese. The latter two can be easily run in parallel in the field, while Scott's method can be used as a further check where laboratory facilities are available.

Nitrites

Prior to the extended use of ammonia in water treatment few people ever gave serious consideration to the possibility of nitrites giving a false color with O.T., for the simple reason that chlorine and nitrites can not exist together as such. Nitrites sometimes do exist in deep wells, but where chlorine is applied enough is used to oxidize all nitrites to nitrates. Chloramines, however, are not strong oxidizing agents so that they and nitrites may exist together. This is particularly likely to happen in swimming pools either of the indoor or outdoor type. One pool in New Jersey, for instance, showed residuals consistently, but the bacteriological results seemed to grow worse and worse. Careful examination showed that this residual was due to nitrites. Outdoor pools, particularly those having earth bottoms, are usually none too clear. Where chloramines are used under such conditions attempts to correct for niritres give such erroneous results that it is usually advisable to test for nitrites directly with sulphanilic acid and alpha-naphthylamine acetate.

Scott shows the following values for apparent chlorine in the presence of nitrites (figures in p.p.m.):

Nitrite nitrogen.....	0.05	0.1	0.2	0.3	0.5	1.0
Apparent chlorine.....	0.005	0.01	0.015	0.02	0.04	0.07

Since very few natural waters contain more than 0.10 p.p.m. nitrites and very few of us can read residuals closer than 0.01 p.p.m. we really do not have very much to worry about on this score. However, where nitrites are present in amounts greater than 0.100 p.p.m. we are now able to lessen the error by using Scott's modified reagent in the dark. This procedure will give results which when combined with bacteriological tests should give very satisfactory results.

Organic matter

Practically all editions of Standard Methods speak of the interference of organic matter in making the test for residual chlorine. Since organic matter usually acts as a reducing agent, this is rather hard to explain except that iron is usually combined with it to make organic complexes. It has been pointed out above that iron in combination with organic matter reacts with ortho-tolidine with greater facility than does iron alone. Furthermore, concentrated algae will many times give a deep yellow to O.T. while the filtrate therefrom will not. I have seen this reaction repeated in several different waters and therefore have the feeling that perhaps the coloration of ortho-tolidine by organic matter is due to iron or manganese complexes and not by the organic matter itself.

Starch iodide

Scott (9) shows that starch iodide is much more delicate than has heretofore been appreciated. For instance, he shows "that in natural, non-acid waters neither nitrites nor manganic manganese produce false tests. In acid solution both compounds simulate free chlorine as to color production. It was noted that false starch tests developed with manganic manganese only at a pH more acid than 4.2 and with nitrites at a pH more acid than 4.0."

Decreasing the temperature and increasing the amount of the indicator solution, increases the sensitivity of the starch-iodide method to 0.02 p.p.m. This is considerably more sensitive than is usually considered and suggests a method whereby an operator may demonstrate to his own satisfaction that the values he is recording from day to day are true and not false residuals. However, in spite of these improvements, starch iodide cannot be used dependably for quantitative estimation of chlorine.

MODIFIED O.T. FOR REDUCTION OF NITRITE AND FERRIC IRON INTERFERENCE

Scott suggests that a modified ortho-tolidine reagent (standard O.T. solution 50 cc., distilled water 35 cc. and concentrated hydrochloric acid solution 15 cc.) will materially reduce errors due to nitrites and ferric iron. We have found in certain cases, where too much acid has been used with ortho-tolidine, that the color fades very rapidly producing low results unless the readings are made immediately after the addition of the indicator. In spite of this the modified reagent

should be useful when used with discretion. Already ortho-tolidine of around 20 per cent acid concentration has been employed very successfully in certain sewage work.

MECHANICS OF O.T. TEST

Broadly speaking, the test is extremely simple. It can be carried out without complicated apparatus and with a minimum of figuring. Really nothing could be simpler than the matching of one color with another. But when the fine points of the test are studied there is plenty room for disagreement. One of the reasons is its extreme sensitivity. Indeed there are very few tests in ordinary chemical work that estimate as closely as 0.01 p.p.m. which is equivalent to one hundredth of a milligram or one hundred thousandth of a gram per liter.

A point which most of us know, but which is often overlooked is that the A. P. H. A. dichromate standards must be viewed through a 300 mm. depth, otherwise, depending upon the depth, errors ranging up to as much as 50 percent will be obtained. This may be demonstrated by preparing a chlorine solution which will match a 0.50 dichromate standard when viewed down through the full 300 mm. depth. The colors of these same tubes will not match at all when viewed from the side where the liquid depth is about 20 mm. This is because the color characteristics of the dichromate and ortho-tolidine are not the same. When Muir and Hale dichromate standards are used, they should be viewed through a depth of 240 mm. These are the depths at which the color characteristics match most closely. All commercial sets are compensated for these differences.

The color of the glass in the tubes may make a difference as great as 100 percent especially in the lower values on the order of 0.01 to 0.20 p.p.m. It is very difficult to get tubes matched as to color. The shades most usually encountered are light green, light blue, a reddish brown, and one that is almost black.

North daylight is best and even then when very accurate results are desired days when the sun is alternately obscured should be avoided.

Position of the tube is very important. When close readings are desired a higher and lower standard should be held on either side of the unknown and their position should be shifted frequently.

One should not look at the tubes for any length of time because the eyes soon become color fatigued with the result that the shades all blend together resulting in hopeless confusion of the observer.

Turbidity and natural color of the water of course must be compensated for which is impossible when the long tubes are used. All of the commercial sets overcome this error by placing a tube of the water under test without the indicator between observer and standard.

SUMMARY

Experience and research tend to prove that ortho-tolidine is an excellent indicator for residual chlorine. Knowing that the color is produced either by oxidation or diazotization, one need no longer be surprised when a color is produced in the absence of chlorine or when higher chlorine residuals are obtained than the amount of chlorine added or when colors are produced after the lapse of an hour or so, or where B. Coli are discovered growing in the presence of what had been supposed to have been an adequate residual, for explanations and correction of these difficulties are now available. It is well established that the yellow coloration of ortho-tolidine by chlorine depends upon a low pH value. Hence the acid solution of the reagent. Nitrites in water and the lignocellulose in wood produce the same color. Other substances than chlorine such as manganese and iron oxidize ortho-tolidine producing a yellow color.

It is not always necessary to apply all the corrections to any one water for it is seldom they all appear together. It rather falls upon the operator to know the water with which he is dealing. For instance why should one worry about nitrites when he knows they are absent? This suggests periodic checks for iron, manganese and nitrites and a careful interpretation of suitable correction methods as the individual case demands.

When dealing with strange waters on which no analyses are immediately available, slowly developing ortho-tolidine colors after 20 or 30 minute intervals should warn of danger. Until accurate analytical data are obtainable it would be well to consider the color developing within 10 minutes for chlorine alone or within 20 minutes where chloramines are present as probably the most accurate measure of the true residual.

The amounts of interfering substance present should dictate the course of action for the determination of true residuals.

In general, corrective measures should always be taken wherever manganese is present. Since all methods appear to have a certain degree of error it would probably be well to apply not only the boiling off method, but also the magnesium sulphate and two sample

methods. The one giving the closest to an average of these could then be adopted for routine use with an occasional check with the other two.

Small amounts of iron, 0.3 p.p.m. or less, do not appear to affect the results greatly, but where greater amounts are present the errors may be lessened by using the modified reagent. Where really large amounts of iron, more than 1 p.p.m. are present sodium hydroxide has been used to remove the iron leaving the supernatant in such shape that the available chlorine may be readily determined.

Nitrites up to 0.100 p.p.m. give little trouble and luckily the nitrite content of the average water seldom exceeds this figure. Chlorine and nitrites do not exist together, but chloramines and nitrites may. Furthermore if chlorine were added to a water containing large amounts of nitrites the chlorine might be all used up with the remaining nitrites giving a yellow color to O.T. which could easily be mistaken for chlorine. Hence, the necessity of testing for nitrites by standard methods. The best thing to do in such a case is either to add enough chlorine to eliminate the nitrites or to alter the operation to prevent their formation. It follows then that there is little excuse for confusing nitrite interference with true residual chlorine. Chloramine residuals, however, may be confused, but the error may be materially reduced by taking advantage of the modified reagent and the development of the color in the dark.

The recognition then of the limitations of ortho-tolidine and the observance of certain refinements in the manipulation of the reagent should give results leading to a minimum of bacteriological control or at least to a more positive and dependable control than is possible by bacteriological methods alone.

I have drawn freely from R. D. Scott papers on "Determination of Residual Chlorine," in the preparation of this paper. They are perhaps the most comprehensive works on the subject to date.

(*Presented before the Four-States Section meeting, April 11, 1935.*)

REFERENCES

(1) Ellms and Hauser: Jour. Ind. Eng. Chem., 5, 915, 1913.
(2) Wolman and Enslow: Jour. Ind. Eng. Chem., 11, 209, 1919.
(3, 5) Tarvin, Todd and Buswell: Jour. Amer. W. W. Assoc., 26, 11, 1934.
(4) Cross and Bevan: "A Text Book of Paper Making."
(6) R. D. Scott: Jour. Amer. W. W. Assoc., 26, 9, 1934.
(7) Buswell and Boruff: Jour. Amer. W. W. Assoc., 14, 385, 1925.
(8) R. D. Scott: Jour. Amer. W. W. Assoc., 26, 5, 1934.
(9) R. D. Scott: Water Works and Sewerage, February, 1935.

AERATION OF WATER BY AIR DIFFUSION

By Frank C. Roe

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Aeration of water has been accomplished in many ways since the earliest days of water treatment. Cascades, splash trays, coke beds, spray nozzles, air lift (pumping) and aspirators, are among the better known methods or types of equipment. Diffusion of air through perforated pipe has also been used.

Quite recently water aeration by air diffusion through porous plates and tubes has been practiced with success. This is probably an outgrowth of and due to the improvement in sewage aeration by the same method, which has been the standard for some years.

Because there is a dearth of technical literature describing the diffused air method of water aeration, the writer has endeavored to assemble design and operating data on existing plants. To do this a standard data sheet form was sent to the superintendent or consulting engineer of each of the eight known plants in operation. Replies were received from six, which form the basis of this paper.

Known installations of aeration units utilizing air diffusion are located in the following places:

St. Paul, Minnesota (perforated pipe)
Brownsville, Texas
Petersburg, Indiana
Ft. Atkinson, Wisconsin
Victoria, Texas
Huntingburg, Indiana
Carlinville, Illinois
Winchester, Illinois
Ontario, Oregon (under construction)
Reading, Pennsylvania (under construction)
Ft. Howard Paper Co., Green Bay, Wisconsin (under construction)
Crown Point, Indiana (under construction)

AERATION OBJECTIVES

Among the existing air diffusion plants there is no one single purpose of aeration which is common to all. (1) Release and reduction

of carbon dioxide and other entrained gases, (2) reduction or removal of odor and taste, (3) oxidation of dissolved iron and (4) mixing of chemicals, are accomplished in the various plants.

Aeration by air diffusion is often selected for two or more purposes of which mixing of chemicals is likely to be one. The advantage and economy of combining an aeration chamber with a mixing chamber is obvious. Where both are required, the air diffusion method is especially adapted.

DESIGN

Only simple structures and equipment are required for aeration of water by air diffusion. Introduction of air from blowers through standard size diffuser tubes suspended in a tank of the spiral flow

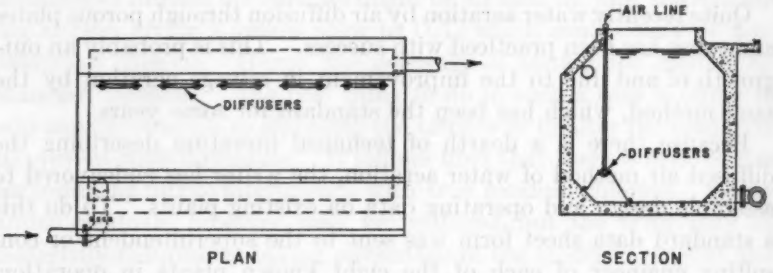

FIG. 1. PETERSBURG, INDIANA. WATER PURIFICATION—AERATION AND MIXING

(Charles H. Hurd, Consulting Engineer; Indianapolis, Indiana)

design, is a commonly used and economical arrangement. Under some conditions it may be more practical and economical to utilize diffuser plates in the bottom of an aeration tank or in channels.

Analysis of the data sheets of existing diffused air installations indicates that an aeration period of 15 minutes is normally adequate to accomplish all objectives, including proper mixing of chemicals. Iron is precipitated in a shorter time, but for release of entrained gases and mixing less than 10 minutes is not recommended. More than 20 minutes is probably unnecessary.

Tank cross sections may be approximately square as at Petersburg, Indiana (fig. 1), or with a maximum width up to two times the depth. Surface area is not highly important in water treatment from a functional standpoint, however, as some consider it to be in sewage treatment. It is believed that the "sweeping" action of rising air bubbles is most beneficial for removal of CO_2, etc.

In small plants (2 m.g.d. or less) the square cross section is probably best adapted in order to secure any appreciable length. In large plants, wide tanks are obviously more economical in construction cost.

Depths from flow line may vary to suit conditions, but less than 9 feet is not recommended. More than a 15 foot depth is without apparent advantage and would result in an increased power cost.

The type of agitation provided by air diffusion through porous mediums inherently eliminates short circuiting. For this reason it is unnecessary to provide a distribution baffle in front of aeration tank inlets unless extremely high velocities exist.

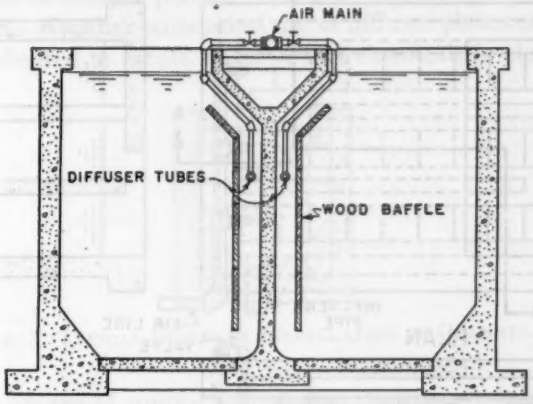

FIG. 2. CROSS-SECTION OF BROWNSVILLE UNIT-ELEVATED DIFFUSERS
(Henry E. Elrod, Consulting Engineer; Houston, Texas)

Aeration tanks of the types indicated by cross sections of the Brownsville, figure 2, and Petersburg units respectively, are both in successful use. The baffled type with elevated diffuser tubes as constructed at Brownsville is the basis for a patent issued to Mr. Henry E. Elrod of Houston, Texas. This is being offered by the Link-Belt Company as its Aerator-Mixer. This scheme obviously permits operation at lower air pressures than the plain tank design.

The plain tank design, as constructed at Petersburg, with tubes near the tank bottom is simpler than the baffled design even though it requires higher operating pressures. Under conditions of low power cost and with certain types of blowers, the cost of operating the plain type at higher pressures than required for the baffled unit, may show it to advantage when considering the extra equipment required for the baffled type.

A single row of diffuser tubes from end to end of aeration tanks provides adequate diffuser area for introduction of the quantity of

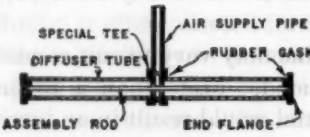

FIG. 3. STANDARD DIFFUSER TUBE ASSEMBLY

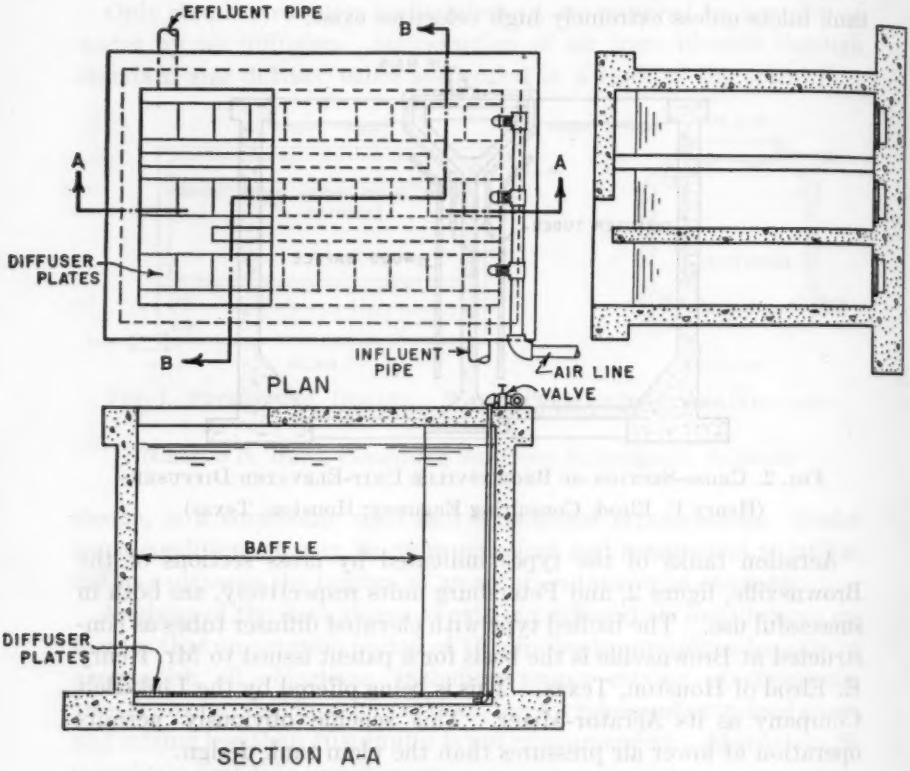

FIG. 4. ONTARIO, OREGON WATER FILTRATION PLANT. PLAN AND SECTIONS OF AERATOR

(L. R. Stockman, Engineer)

air required. The most satisfactory method of installing or assembling diffuser tubes is shown by figure 3. The special tee, end flanges,

assembly rod and gaskets were developed by the Link-Belt Company and are sold by them. These fittings are given a corrosion resisting treatment.

Diffuser plates may be used instead of tubes in the spiral flow type of tank design. Ordinarily such a design is best adapted to plants of multiple units where operation of one aeration unit can be interrupted for servicing of diffusers without disadvantage. The only advantage of individual tube assemblies over plates is that they can be disconnected at a union above water and lifted out of the tank while it remains in operation.

The aeration unit designed for Ontario, Oregon, figure 4, offers an ingenious use of diffuser plates. It is a three pass "around the end" baffled type. Another economical use of diffuser plates can be made where it is desired to aerate water or mix chemicals in channels con-

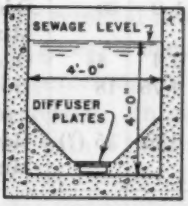

FIG. 5. TYPICAL DIFFUSER INSTALLATION IN CHANNEL

necting the source of supply with flocculation or coagulation units. (See figure 5.)

Diffuser plates or tubes of the type usually employed for aeration of sewage are suitable for aeration of water. A medium-high grade of 35 to 40 permeability (based on the standard method of rating) is recommended. Diffusers of this grade have a low pressure loss, resist clogging and discharge bubbles which are not noticeably larger than are discharged from finer grade diffusers.

The data sheets received indicate that air requirements may vary from $\frac{1}{20}$ to $\frac{1}{10}$ cubic foot per gallon of water treated. Assuming an average air requirement of $\frac{1}{14}$ cubic foot per gallon of water aerated, a plant capacity of 1 m.g.d. would require an air blower capacity of 50 cubic feet per minute.

Centrifugal or positive displacement type blowers are recommended. Air filter equipment should be provided in connection with blower intakes.

COST

The initial cost of the simple structures and equipment described in the foregoing section compares most favorably with the cost of other methods of aeration. The total equipment cost *installed* is

TABLE 1

Operating data

PLACE	YEAR BUILT	CAPACITY (M.G.D.)	PERIOD OF AERATION (MINUTES)	OBJECTIVE	AIR REQUIREMENT (CU. FT. PER GAL.)	POWER REQUIREMENT (KW. PER (M.G.D. CAPACITY)
St. Paul	1922	23.0		Odor removal Mixing	0.005	
Brownsville	1931	4.0	14	Gas removal Mixing	0.07	0.48
Petersburg	1933	1.0	30	Odor removal Mixing	0.10	1.46
Huntingburg	1934	0.4	8½	Odor removal	0.15	0.75
Ft. Atkinson	1934	0.75	18	Iron and odor removal	0.16	1.87
Victoria	1934	0.52	35 (?)	Gas removal Mixing		

TABLE 2

Analytical data (p.p.m.)

PLACE	CARBON DIOXIDE		HYDROGEN SULPHIDE		IRON		ODOR	
	Raw	Effluent	Raw	Effluent	Raw	Effluent	Raw	Effluent
St. Paul*	3.8	7.2			0.06	0.06	2.5	0.4
Brownsville	2.43	1.71						
Petersburg	3.0	3.0			1.15	0.03		
Huntingburg	No data available							
Ft. Atkinson	16.0	7.0	Considerable	Nil	0.35	Nil	Some	Nil
Victoria	11.0	4.0	19.0	0.0			Some	Nil

* Manganese removal 0.08 p.p.m. to nil.

normally less than $500.00 for a one m.g.d. unit which cost obviously decreases as the capacity increases.

There is no extra lift of water required for aeration by air diffusion as the flow line of units is at the flow line of subsequent sedimentation

basins and filters in the usual plant layout. Therefore, the only operation cost is for blower power requirements. The power requirement per m.g.d. capacity varies considerably. For purposes of estimating, an average figure of 1.0 kilowatt per m.g.d. capacity can be used.

RESULTS

The tabulation of data as received for the preparation of this paper should be used as a record only and not as a basis of comparison. It is apparent that local conditions, requirements and general objectives control the basis of original design and the results to be obtained. No two installations are alike in every respect. The data are presented in tables 1 and 2 with the foregoing considerations in mind.

SUMMARY AND CONCLUSION

1. Satisfactory reduction or removal of entrained carbon dioxide, hydrogen sulphide and other gases, is accomplished by the diffused air method of aeration. The percentage of removal increases greatly as the raw water content increases. In general, carbon dioxide removal will exceed 75 percent with a high raw water content. Hydrogen sulphide removal is more complete.

2. Odor reduction and some improvement in taste, depending on the quality of the raw water, can be claimed through the use of diffused air aeration. Odor in particular, as caused by aromatic oils, may be eliminated sufficiently by this method. Water having very strong tastes and odors should be given ammonia-chlorine treatment or equivalent.

3. Air diffusion will quickly and thoroughly convert the iron content in water to its hydroxide form for subsequent easy removal. This also applies to oxidation of manganese which so often occurs with iron.

4. Mixing of chemicals by air diffusion is positive and thorough. The velocity of the water in its flow across the bottom of aeration chambers is sufficient to prevent accumulation of sediment or floc. There are no unagitated or unaerated dead spots. The floc created is normally large and heavy. The rising stream of air bubbles and the continual overturning motion of the water does not break up the floc.

5. The air diffusion method of pre-treatment provides an economical solution where both aeration and chemical mixing is required.

6. Aeration units utilizing air diffusion can be operated in freezing weather without ice troubles.

7. The degree of aeration can be varied easily to meet fluctuations in raw water quality by varying the amount or quantity of air diffusion.

8. Practically no head of water is required to operate diffused air units which permits their construction at the same elevation as other elements of a plant. This simplifies design and decreases construction cost.

9. The installation and operation costs of diffused air plants are comparatively low.

ACKNOWLEDGMENT

The writer wishes to acknowledge the courtesy and coöperation of the following engineers and superintendents who provided general information, sketches, photographs and operating data for the preparation of this paper.

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(Presented before the Illinois Section meeting, April 11, 1935.)

A PLANNED FUTURE FOR THE DISTRIBUTION SYSTEM

By C. R. Bird

(*District Engineer, The Pitometer Company, Engineers, Detroit, Mich.*)

In these days more attention is being paid than formerly to planning for the future of communities. Zoning plans which regulate the type of development, and master plans which outline street changes and other means to facilitate traffic movement and other betterments are becoming more and more common. The public utilities, especially the telephone companies, are constantly making studies so that their facilities may be adequate to care for future demands. Why is it not as important to plan in the same way for that most vital of utilities, the water system?

For years most water distribution system extensions have been designed largely by the good old method of the red pencil and blue print. New houses have needed water where there was no main to supply them, and a line drawn on the map from the nearest existing main has indicated the pipe which was considered large enough for the immediate purpose; with very little thought of what might be necessary for possible future development in the area beyond, or the effect on the area formerly supplied.

In some instances a forward looking official has made studies from which a tentative program was drawn up; but changes of administration often have made these plans of little use, since they were perhaps more in the nature of personal opinions covering small areas rather than well rounded studies of the entire problem based on full knowledge of the facts. Thus we find large mains, started with the idea of future extension to a proper source, now carrying only a small fraction of their capacity, their original purpose forgotten, and their cost of construction to a large extent wasted.

With private companies whose franchises have been in danger of recapture by the communities served it is not surprising to find that a hand to mouth method of additions has been in effect. It is not unusual when a city has taken over a private company's plant to find that heavy capital expenditures are necessary to bring the system

to a point of reasonable efficiency. Some of the stronger companies, however, have not seen the need of a proper program for the future, or have been unwilling in the past to incur the comparatively small yearly expense of insuring an adequate plant for the future, when cost and difficulties of additions would in most cases run far higher, due to permanent pavement repairs, etc.

Municipally owned plants have an added incentive to hold down the costs, since higher water rates or taxes may be necessary if the full value of the water service and fire protection is to be obtained.

As a result of these false economies we find the water works orphan clad in outgrown garments, and often robbed of what little legitimate profit is possible which should be used for betterments, to eke out other departments, to build hospitals, to supply free water to schools and parks and the like; until the time comes when there is a cry of water shortage, low pressures and poor service generally. If these conditions are remedied before a disastrous fire occurs it is fortunate.

We wish to emphasize at this point that these troubles are not the result of neglect of today or yesterday or last year. They are the cumulative neglect covering many years and could have been avoided if a properly considered program of construction with a master plan had been adopted so that additional feeders and adequate distribution mains could have been installed as they were needed. If such procedure had been adopted the yearly profits of the water department would have been used, where they should have been, to improve the water service, and no very heavy burden would have resulted at any time.

In these alphabetical days many of the smaller communities are installing entirely new water systems and these are necessarily designed by engineers who can, without considerable increase in the cost, make some provision for the future. The older communities are not so fortunate. The pipes are already installed over a considerable portion of their area, and often are not of sufficient size for the work they must do. Some of the feed mains may carry only a small percentage of their capacity, while others are overloaded. Plenty of water may be available at one point where the demand may not nearly equal the ability of the mains to supply the needs, while at other points the demand, either ordinary or exceptional, may draw down the pressures to the danger point.

Such a study of the distribution system as we have mentioned, with a carefully worked out program for the future, will eliminate or avoid these troubles.

It may be claimed, and often is, that a program of such work is not much better than a guess; but every engineer knows that a prediction based on solid facts of present and past conditions is far more likely to cover the future conditions than a personal opinion which is unlikely to have the whole picture as a background.

That great hydraulic engineer, John R. Freeman, is said to have made a study of past and future conditions of Providence, Rhode Island, in the middle 1870's to cover fifty years of future development. It was only after 1920 that the system began to feel the need of further planning; and a considerable part of this need was due to the construction of a new reservoir, with consequent changes in the supply in the city. This is an exceptional case and nowadays we do not consider it advisable to plan for such a long period, but it shows what can be done by careful analysis of all the facts in the engineer's possession.

BASES FOR FUTURE DESIGN

It is perhaps unfortunate that we are not endowed with the gift of prophecy, but lacking that gift there are certain basic facts which can and must be used in designing for the future. These are:

1. The past growth of the community.
2. The present density and character of occupation.
3. The past and present water consumption of the system and its component parts.
4. The fire requirements.
5. The carrying capacities of its present feeders and various sections of the distribution system.

For the population studies we have the decennial census figures covering several decades in the past; and in the larger cities the ward figures also for two or three decades. In some states a census is taken between the U. S. Census years. From these by use of various methods of computation a reasonably close estimate may be made of the probable population twenty-five years in the future.

A field study of the density and character of occupation is needed as a guide to future development and in this study a zoning plan is of great value if such a plan has been adopted. At the same time important factory development will usually be along the lines of transportation, whether by railroad or water. Parks and cemeteries will in most cases remain unchanged in the future, although both may sometime be enlarged. Golf courses and country clubs may also

remain unchanged, but some thought should be given to their financial standing and plans for the future, since it is possible if they are abandoned, that a considerable building development may take place.

Usually the records of the total water consumption of the system are available and from these an estimate can be made of the future water requirements for the whole city.

The fire requirements are quite fully defined by the National Board of Fire Underwriters according to the size of the city. Flow tests from hydrants will give conditions approximating fire needs, and properly arranged will show a clear picture of the present fire protection, both for the feeders and for the gridiron system.

The carrying capacities of the feeders are of great importance, and definite knowledge is needed as to their interior condition. Loss of head tests are necessary to give this information and these should be made simultaneously with measurements of velocity in the pipe. Such measurements should be based on actual gauging of the velocity in the pipe rather than by hydrant flow measurements or other means which may or may not give the true pipe velocity, and which may not be much better than a mere guess.

Another point in which an estimate—or to speak more plainly a guess—is often used, is the amount of water supplied to the various sections of the city. It is very rarely the case that the per capita consumption of a system is the same or even approximately the same throughout the system. Absurd results may easily be obtained by using such an assumption, certain sections being greatly over-piped; while others, where far more water is required, may be so completely neglected as to greatly increase the fire hazard. Hence actual measurements of water consumption in small districts throughout the city in combination with census figures covering the same areas are important to obtain the per capita consumption in each section. This gives a logical basis upon which to predict the future need in each of these areas.

With these basic data the work of design divides itself naturally into two parts. First, the feeders must be designed to care for the present and future needs of the community. It is not sufficient to say that just so much main capacity is needed now and that just so much more will be needed by 1960. To arrange a program of construction it must be known quite definitely just when a certain section will need the support of a proposed feeder. From a proper study of the data this can be ascertained within close enough limits so that

the main can be laid in time to anticipate the need; and at the same time the interest charges can be saved which would be required if it were laid too soon or the embarrassment can be avoided of having insufficient supply when the need has fully developed.

The other part of the design work has to do with the distribution gridiron, and here there is less opportunity for designing and greater field for the exercise of good judgment and horse sense, combined with a careful study of the local conditions.

In many systems, perhaps we might say in most systems, a considerable percentage of 4-inch pipe will be found. As is well known, this small pipe is of little value in fire protection and it is safe to say that water works executives in general would feel better if it could be entirely eliminated. Since the cost of replacing it all with larger pipe would be prohibitive a study is made to see what can safely be allowed to remain. Certain parts will undoubtedly be replaced or paralleled by the proposed feeder mains, the routes of which, as far as possible, follow unpaved streets and those where the present pipes are inadequate. Often the study will show that, by changing the hydrant leads to an intersecting main, a short length of 4-inch pipe may remain in place as a domestic supply; and that it will also serve a valuable purpose as a cross connection between two larger pipes. Unless some such arrangement can be made or the block is very short, it will usually be necessary for fire protection to replace or parallel the 4-inch with a larger main. Since there are generally so many of such 4-inch mains that the cost of replacement would be very great, reconstruction must necessarily be spread over as great a time as possible so that the more important needs may be met. Therefore, some selection must be made of the most important for early change, and so on to the less important, for inclusion in the different divisions of the program. This is not difficult for the former case, because the more thickly settled a district and the greater the fire hazard from cheap construction and wooden buildings, the more important it is to get adequate protection as soon as possible. The latter group presents greater difficulties to arrange in a program. It is manifestly impossible for anyone to say that Twentieth Street between Avenues A and B will have a fire requiring more water than can be supplied by the present small main, earlier than the block of Thirtieth Street between Avenues C and D where the conditions are similar.

About the only thing to do, after a careful study has been made of all the conditions and as many of these small mains as possible have

been allocated in the program, is to lump the remainder and to take care of them as fast as finances allow or paving construction demands. In either case the master plan will show what is needed in these blocks and the construction can be carried on to suit conditions as they arise in the future.

The program itself is designed to cover requirements for twenty-five years in the future. It is divided usually into seven parts: First, immediate needs intended to include the most pressing of the recommendations to give adequate supply for consumption and enough of the fire requirements to make the community safe except in case of an exceptionally bad fire. Second, five periods of five years each, covering the advancing needs of the twenty-five year period, and also completing the fire protection of the community. Third, a schedule of mains which should be replaced but for which no reasons can be found to assign them to any particular date. In each of these divisions are given the location, size, approximate length and approximate cost of the improvement.

It is, of course, inevitable that changing conditions will somewhat alter the program, since our designs are based on probabilities which may not eventuate. Since a considerable part of the capacity of any water system must be designed for fire protection, an error in the population estimate, even of some magnitude, will make little difference in the sizes of mains to be laid. However, where area of any size, now sufficiently supplied, does not develop as rapidly as expected, it will be possible to defer portions of the proposed construction to a later period. On the other hand, an area may develop faster than expected for some reason impossible to foresee at the present time, and a proposed improvement may need to be advanced in the program.

From the changeable nature of human affairs the program is not and cannot be a hard and fast rule of procedure. If all the variables were constants in the equations the program could be used at all times throughout its life as originally scheduled. But paving and repaving construction, and other changes in the fundamental data cannot always be predicted.

A recent number of Water Works Engineering contained an article presented at the meeting of the Southwest Section of This Association entitled "Planning a System 25 Years Ahead" by Mr. Albert R. Davis, Superintendent of the Water Department of Austin, Texas, where early in 1928 such a survey was made and program arranged.

In this article he states that a bond issue of $4,250,000 included $2,000,000 for street improvements but none for the Water Department. This meant that thirty-five miles of streets were to receive permanent paving. He says:

"Right here is where I learned to appreciate what a distribution study means to a water superintendent. . . . . It gives a certain confidence to have a well defined plan to follow, for with it goes the feeling that the job is being done properly and will not have to be redone before the concrete hardens. . . . . After five years of following a 25 year plan the City of Austin has its first concrete pavement to break to install something which should have gone in before the pavement was laid."

We can close this paper in no better way than to quote the last paragraph of this article by Mr. Davis.

"In conclusion, it may be pertinent to say that a study of a distribution system with a well defined plan for a period of years is one of the most progressive steps that is possible to be taken. The writer has found in discussing construction work with the ordinary layman, that whenever it is explained that construction work to be done must conform to a well defined plan there is a ready response and commendation on their part for what is being done. So if the layman can see the feasibility how much more should a water works superintendent be able to realize the value of a distribution study."

(*Presented before the Indiana Section meeting, March 7, 1935.*)

THE STATE'S INTEREST IN OUR WATER RESOURCES

By W. H. Frazier

(*Sanitary Engineer, Conservation Department, Indianapolis, Ind.*)

Indiana must go a step further than ever before in the centralization of authority if the water resources of the state are to be conserved and properly serve the needs of her citizens. It is never popular to suggest further centralization of authority in the state at the expense of the individual, the municipality or of business, but such a step is ahead of us if we are to accomplish our purpose.

Debatable as such action may seem, there can be no debate over the fallacy of local control of water resources as demonstrated during recent years. We have witnessed the plundering and waste of other natural resources. Our forests were destroyed to make way for the plow that now rusts because the fertile soil has been washed away; the storehouse of oil and gas has been drained for all time and we have awakened, the heirs of a spendthrift generation, with only a pittance of the natural resources that might have been ours.

Our water resources have not escaped, but we have an opportunity to restore and conserve them, regaining some of the benefits which they alone make possible. Past experience shows that such effectiveness can be accomplished only through united effort, actuated by a thorough appreciation of the statewide nature of the problem and its influences upon the daily life of every man, woman and child.

A year ago, I urged your consideration of the problem of pollution as affecting the water resources of the state; today I am suggesting the need for some one agency in which can be vested full authority not only to investigate, and study, but to act for the best interests of the state as a whole upon the important subject of water resources. I invite your serious consideration of this subject.

We know something of the part that Indiana's water resources have played in the development of the state; how the rivers and streams were the highways for the red man and for the white man when he pushed southward from the great lakes and westward over the Alleghanies; how the early settlements were established along

these streams and the importance of these streams to the pioneer as a transportation system and a source of power to turn his mill. Water was a determining factor in the establishment of every pioneer home either along a stream, near a spring or at some point where it could be obtained easily for the family and its livestock. Towns and villages were located at points where water was plentiful.

We are familiar with the subsequent history of Indiana's waterways, the expenditure of millions of dollars for canals which were discarded for the railroad, and with the expenditure of other millions for the drainage of swamps and lowlands.

Indiana entered upon an era of industrial development with its rapidly growing population grouping into larger and still larger cities. With this era came the problems of water supplies for both the industry and the city and the attendant problem of disposing of the waste. At first, the answers were simple, the water came from a stream or from wells and the waste was dumped into the stream to be carried away.

These conditions were our inheritance. We had thousands of miles of rivers and streams polluted by the waste of industry and the city; we had the growing task of providing an uncontaminated supply of water or of so treating what water was available, so that it might be reasonably free from disease and suitable for human needs. We had thousands of acres of land from which the fertile top soil had been washed, no longer profitable for agricultural uses. We had other thousands of acres which were suffering from a lowering of the water table as much as 40 feet. We had floods at one time of the year and stagnated streams at another.

We had streams so laden with waste that disease affected the bather, killed livestock and turned from a place of enjoyment and recreation to open sewers. We had a state in which 40 percent of the population had no sewerage facilities, in which few cities made more than a pretence of treating their sewage before it was dumped into some stream to pollute the waters for miles below, and in which at least 24 communities had water supplies questionable from a health standpoint. We had lake levels shrinking to a fraction of their former depth and conditions existing in many of the streams that killed all but the lowest form of water life.

Such were the conditions when the control of water resources was regarded solely as a local problem. Of course, such conditions did not and do not prevail in every section of the state. Some cities

have installed and maintained sewage disposal systems of the most efficient types; some industrial firms have shown their regard for the health and welfare of the people by so treating their waste that it does not cause pollution of the streams into which it finally finds its way.

For a score of years, attempts had been made to give the state authority to regulate the turning of waste, both industrial and municipal, into the streams. Some progress was made, but little impression was made. Cities continued to turn their waste into the streams, creating unhealthy, offensive conditions for other cities downstream and industries followed the example set by the cities. Surveys have shown that approximately fifty percent of the pollution of Indiana streams today is due to the cities dumping their sewage without adequate treatment.

During the present session of the General Assembly, legislation has been enacted and approved by the Governor which makes possible an effective campaign against continued pollution of Indiana streams. It was enacted after selfish interests had waged a bitter fight to amend it so that it would leave them free to continue their pollution of the streams without regard for the general health and welfare.

They charged that this law set up despotic powers, that it would drive industry from the state and would bankrupt cities where present sewage disposal systems are inadequate. The law does grant to the Administrator of its provisions, The Department of Commerce and Industry, authority exceeding that provided in any former measure of its kind. But without proper centralization of this authority, there would have been no means of attacking the problem of pollution. It has never been the policy of the state or any of its agencies to drive legitimate industry from the state or to impose undue burdens upon any municipality or group of individuals.

NEW LEGISLATIVE ACT

The Act provides for a Board of three members. These members will be named by the Governor, each to serve terms of four years without additional compensation. It shall be the duty of these board members to enforce the provisions of the Act. It becomes the duty of this board to determine what constitutes pollution, and after determination that a stream or part of a stream is or is about to be polluted, to order the condition corrected. Provision is made for an appeal from the order of the board to the local courts and in the case

of municipalities, alternative methods are provided for the financing of such improvements in a sewage system as may be required.

There will be, I am certain, no disposition on the part of the board when named to administer the provisions of the law without a thorough study of the cause of each case of pollution and first seeking coöperation of the industrial plant or the municipality responsible. The penalties of the law will be enforced, but only when all other means have failed.

Pollution of Indiana's streams cannot be ended immediately under any condition or situation. It will require many years to seek out and correct all the conditions that cause pollution. A method of treatment for the waste from different plants must be worked out. Each case presents a different problem, and what will work in one may not work in another. A number of the industrial firms of the state have already demonstrated their willingness to coöperate in ending pollution and are installing equipment for the treatment of the waste from their plants before it goes into a stream.

Several of the cities of the state have acted during the past year, to give adequate treatment to their waste and end their pollution of the streams. In this they have been aided by the Federal government with Public Works funds, and completion of their projects will be a step toward the elimination of pollution in Indiana.

To attempt the ending of pollution under any plan of local control would have been futile. Realization of the menace of pollution to the health of Indiana's citizens and the necessity of state rather than local control, led the members of the General Assembly to enact the new legislation. With such a statute in effect, Indiana has taken a forward step toward control of water resources, but if the greatest good is to be realized from those resources, additional steps must be taken.

CENTRALIZING WATER RESOURCES CONTROL

The next step and the logical one, is the placing of all matters pertaining to water resources in the state agency. This agency should not only conduct a state-wide survey of Indiana water resources and plan for their greater utilization, but should also have the power to see that such plans and programs are observed and made a part of local projects. Such a task cannot be undertaken successfully except on a state-wide basis. What one municipality, one industrial plant, or one individual does with reference to water resources no longer

affects that municipality, plant or individual alone; it affects other cities, plants and individuals in other localities, many miles away.

The water resources of the state cannot be considered within municipal boundaries or county lines, and the time will come within a short time when the entire nation, rather than the state, will be the unit in undertaking this problem. We have witnessed many changes during the past decade or two, and there is no reason to doubt that other changes affecting water resources will not take place during the coming years.

There are many factors to be considered in any study of water resources on a state-wide basis. We are, with the coöperation of the Federal government through the Civilian Conservation Corps, making an effort to control erosion. We are making a beginning in the improvement of streams with the expectation that they will again flow throughout the year, and not for a few months, reducing if not entirely eliminating, the danger and the damage from floods. Flood control is not a matter of building more and higher levees but of re-establishing former conditions at the headwaters to prevent a rapid drain of water.

The efforts being made for the reforestation of large areas of the state and for the establishment of forested areas along the banks of the streams will be of real value in our efforts to reduce the menace to life and property that now exists through much of southern Indiana as a result of the rapid run-off of water from the central and northern parts of the state. This reforestation movement has another value in our consideration of water resources. There is reason to believe that the presence of large wooded areas mean an increased rainfall with less rainfall in areas devoid of forests.

At the present time, two agencies of the Federal government, the National Forest Service, and the AAA, are acquiring several thousand acres of marginal and sub-marginal land in southern Indiana, expecting to create forests which will be open to the people of the state for recreational purposes. Similar areas should be established in the northern section of the state. Although the acquisition of the land would be far more costly in the north, the benefits would be correspondingly large. If we are to maintain our agricultural status, we must retain the water resources in the agricultural sections, or those areas will lose their fertility and their value for production.

Another step in preserving and increasing our water resources would be the control by the state, through its water resources board,

of all lake outlets and spillways, as a means of effectively regulating lake levels. In northern Indiana where lakes abound, lake levels are important to a degree not readily recognized by the central and southern sections of the state. In the past, it has been possible and a fact, that one owner of property abutting upon a lake has blocked efforts of all other property owners to restore a lake level, simply because it meant that the lowered level gave him a small area for cultivation and would be lost if the level were restored.

During this session of the General Assembly legislation has been enacted to change that condition and as a result, a general increase in lake levels will be possible.

In the past years, we have regarded the fisherman and the sportsman, as an eccentric, maybe a good fellow, but a little queer because he occasionally lifted his nose from the grindstone to enjoy a few hours or days in the open. Now our attitude is changing for we have discovered that these fellows spend thousands of dollars every year in the pursuit of their sport and their hobby. These sums are spent for equipment, for travel, for food and shelter and represent for Indiana an almost untouched revenue. We have recognized to some extent the value of our lakes and streams from an agricultural standpoint, but their potential value for recreation is far greater than we have realized. If Indiana can restore the purity of the lakes and streams, it will be an accomplishment or more value to the health and well-being of future generations than any other thing we might do.

I have sought to outline something of the importance to our water resources of pollution and increase their value to our citizens, and to point out hastily the necessity for some plan of unified supervision and control of these resources if they are to be continued.

There will be need for continued supervision of these resources by some authority with more power than that of merely making suggestions or recommendation. We must abandon our former local viewpoint and broaden our vision to encompass the needs and the future of the state. We must accord to water resources their true value, and study to increase that value in the social and economic life of the people.

(*Presented before the Indiana Section meeting, March 7, 1935.*)

ABSTRACTS OF WATER WORKS LITERATURE

FRANK HANNAN

Key: American Journal of Public Health, **12**: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Replacing Old Service Pipe. Eng. News-Rec., **109**: 376, September 29, 1932. Procedure employed by gas company in Baltimore in replacing service pipes is briefly outlined. Small excavation is made at junction of main and service and 4-foot length is cut from latter. New service pipe is then connected to old in house and old pipe is drawn toward main with pulling apparatus, 3- to 4-foot sections being cut from old pipe at excavation and 20-foot section of new pipe added to house end as pulling proceeds. As much as 100 feet of pipe has been installed in this manner without disturbing lawn or shrubbery.—*R. E. Thompson.*

Contraction Joints of Pardee Dam Grouted a Second Time. F. W. HANNA. Eng. News-Rec., **109**: 431, October 13, 1932. Continued cooling and shrinkage of Pardee Dam following first grouting at time structure was completed in 1929 made second grouting advisable to reduce seepage. This regrouting, completed early this year, has been effective in reducing seepage with reservoir full to 0.85 second-foot compared with 2.80 at time of first filling, which followed first grouting. Thermocouples placed about mid-height indicated initial internal temperature of 79°F.: temperature later rose to maximum of 102° and has dropped again to 79° at present time. In first grouting 413 barrels of cement were used and in regrouting, 425 barrels. If further cooling and contraction causes increased seepage, additional grouting will be carried out.—*R. E. Thompson.*

An Improved Method of Feeding Sedimentation Units. Cont. Rec. and Eng. Rev., **46**: 1271–2, November 9, 1932. Improved method of feeding sedimentation units to be known as Sifeed, recently developed by Dorr Co., is described briefly and illustrated. Distinctive features are introduction of influent through centrally-located inverted siphon and overflow of effluent across peripheral weir extending around entire tank. This development has led to new design of Dorr central drive clarifier in which compact driving unit is mounted directly upon top of central column and not on cumbersome bridge-type superstructure formerly employed.—*R. E. Thompson.*

Fire Prevention Week Services to Emphasize Public Responsibility. Cont. Rec. and Eng. Rev., **46**: 1178, October 12, 1932. During past decade, exclu-

sive of forest fires, 407,000 fires in Canada destroyed property valued at over $415,000,000, 3514 lives being lost and 9067 persons seriously injured. In 1931, 49,284 fires were reported, propertly loss being $47,120,000 and loss of life 251. Including cost of insurance protection and of maintaining fire departments and water supplies for fire extinguishing, unproductive expenditure of over $100,000,000 was incurred in Canada during 1931 through fire. Seventy percent of fires in Canada occur in dwellings. Fire occurred in approximately 1 out of every 10 dwellings during past 10 years. Ninety percent of fires are avoidable, being caused by sheer carelessness. In Great Britain, number of fires is less than one-quarter as many. In 1931, fire losses in United Kingdom, with population of 50 million, amounted to $51,000,000, or just over $1 per capita compared with $4.50 per capita in Canada.—*R. E. Thompson.*

Pipe Laid Across Panama Canal by Use of Cable Support. A. L. HERTZ. Eng. News-Rec., **109**: 562-3, November 10, 1932. When it became necessary recently to extend water system of Balboa, C. Z., across Pacific entrance of Canal, installation was completed without obstructing traffic by suspending pipe lengths from steel cable as joints were made up and lowering line to bottom when necessary. Line was of 12-inch cast iron pipe with metropolitan-type flexible joints permitting deflection of about 13°. Channel at crossing is 800 feet wide, dredged to elevation −50, mean sea level datum. Trench was dredged to elevation −60 and slope from canal prism lines to elevation 0 on each side; distance of about 1500 feet. Tidal range is about 18 feet. Total of 127 lengths were laid in 35 hours: maximum speed of eight 12-foot lengths per hour was attained, but average was about 5 joints per hour. On completion, line was tested with compressed air and the few small leaks were easily calked by diver. Procedure outlined.—*R. E. Thompson.*

Sanitary District Blames Delay on Chicago Tax Angle. Eng. News-Rec., **109**: 602–3, November 17, 1932. Lack of funds, a consequence of involved state of tax collection in Cook County, was given by Sanitary District of Chicago as main cause of delay in carrying out program of sewage plant construction in answer to United States Supreme Court's recent demand that it show cause why appropriate steps had not been taken to comply with requirements of court's decree dated April 21, 1930. Reduction in diversion of water from Lake Michigan to annual average not exceeding 6500 second-feet was made July 1, 1930 and has since been adhered to. On July 1, 1932, 31.6 percent of sewage of district was being treated. Non-payment of taxes has seriously interferred with program. Aggregate of $41,105,000, equivalent to 2 years' taxes, were past due and unpaid up to October 26, 1932.—*R. E. Thompson.*

Foundation Design for an Elevated Tank. M. B. MEYER. Eng. News-Rec., **109**: 644–5, December 1, 1932. Details given of foundation design for new 1.5-m.g., radial-cone-bottom elevated tank recently put in service in Evanston, Ill. Tank is 104 feet in diameter and 92 feet high to bottom and has range between upper and lower water levels of 25 feet. The 393 wood piles supporting the 49 concrete piers were driven to depth of 70 feet. Brief data are given regarding settlement observations and strain-gage readings on steel columns

made after first filling and emptying. Average settlement of all piers after water had been in tank 100 hours was about 0.25 inch. Complete recovery occurred in emptying.—*R. E. Thompson.*

Continuous Water Chlorination. Can. Pub. Health J., **23**: 392, August, 1932. Importance of continuous chlorination is stressed. Provision of duplicate apparatus is surest method of avoiding interruptions. Instance is cited of typhoid epidemic resulting from chlorinating apparatus being shut down for period of one half-hour to make slight repair.—*R. E. Thompson.*

Corewall in Rockfill Dam Tilts When Reservoir Fills. F. A. Noetzli. Eng. News-Rec., **109**: 529–30, November 3, 1932. Oued Kébir Dam, of Tunis (Northern Africa) water supply system, located about 50 miles south of city, is rockfill structure, 115 feet high and 1140 feet long on crest, forming 21,000-acre-foot reservoir. Outstanding feature of dam, which was completed in 1925, is hollow vertical corewall with overall thickness of 6.5 feet at top and 30 feet at base. During first 4 years' operation, water level stood at about one-half height of dam. Settlement of rockfill in both up- and downstream sides of corewall was observed, amounting to 2 feet at maximum section. In March, 1929, flood filled reservoir to within 6 inches of lip of spillway, 7 feet below dam crest. Under this load, rockfill settled considerably and corewall suffered downstream deflection of 2 feet 8 inches at top. At several places downstream face of fill started to bulge, pieces of facing rock broke out and leakage developed at base of dam. While dam was being relieved of load, corewall moved gradually upstream but did not return quite into original position. Inspection showed that corewall had been sheared off from cutoff wall and had been bent and tilted downstream, horizontal displacement being 5 inches at base and 2 feet, 8 inches at crest. Dam was repaired in 1930–1 by filling lowest parts of cells of corewall with concrete and anchoring this new mass to cutoff wall and to foundation rock. Cracks in arched upstream deck of corewall and in cross-walls were also repaired and downstream fill strengthened by addition of fill material at toe of dam. Complete description of construction and subsequent repair, by G. Arbelot, appeared in La Revue Industrielle, September, 1931. Lessons to be drawn from behaviour of this dam are discussed. Author doubts if repairs have removed fundamental sources of trouble and have strengthened dam sufficiently to provide as large a factor of safety as is desirable.—*R. E. Thompson.*

Deep Well Water Supply for the Village of Nipigon, Ontario. Cont. Rec. and Eng. Rev., **46**: 1267, November 9, 1932. Brief description of new supply system, which consists of well 70 feet deep, pump house, 150-gallon per minute turbine pump, and 153,000-gallon covered reservoir. At present there are only 26 services. Cost of system was approximately $40,000.—*R. E. Thompson.*

Eight Years' Operation of Passaic Valley Sewer. J. Ralph Van Duyne. Eng. News-Rec., **109**: 440–2, October 13, 1932. Describes intercepting sewer system of Passaic Valley Sewerage Commission, completed in 1924 at cost of

$21,000,000 to correct pollution of Passaic River, and the organization controlling and operating same, and recounts benefits derived therefrom. System, which serves 22 municipalities with total population in excess of 1,000,000, consists of main intercepting sewer, branch sewers, pumping stations, sedimentation basins, and outfall tunnel discharging in middle of New York Bay, 2 miles north of Statue of Liberty. Sludge from basins is discharged from barge into ocean 8 miles beyond Scotland Light. Oxygen dissolved records, which are shown graphically, indicate progressive improvement in river conditions. In 1924, when system was placed in commission, there were 422 industrial concerns discharging wastes into river, or tributaries, and there also remained three serious sewage pollution centers. For several years the Commission endeavoured to eliminate these remaining sources of pollution by persuasive methods, and in most instances the desired results were accomplished. A few very serious pollutions persisted and only after resorting to legal action was final diversion of all sewage and polluting trade wastes from river effected. At present time all such polluting material is either turned into intercepting sewer or is satisfactorily disposed of by means of modern efficient treatment plants, and the once grossly polluted waters of Passaic River have been restored to wholesome, sanitary condition.—*R. E. Thompson.*

Revised Method of Averaging Oregon Precipitation Figures. L. L. HARROLD Eng. News-Rec., **109**: 643, December 1, 1932. Beginning with January 1, 1931, Oregon section of U. S. Weather Bureau changed its method of computing mean monthly and annual records. Formerly, state average was arithmetic mean of all stations having 10 or more years of records. Revised method consists of computing arithmetic mean of 13 section averages. Revision was needed owing to unbalanced condition of state with respect to density of rainfall, etc., and errors due to unequal distribution of precipitation stations. Similar method was adopted in 1927 in Washington.—*R. E. Thompson.*

Tuning Up the Saginaw Water Plant. Eng. News-Rec., **109**: 686–7, December 8, 1932. The excess lime-soda softening and filter plant at Saginaw, Michigan, consists of mechanical mixing chambers, settling basins operated in series, primary basins being equipped for continuous sludge removal, carbonating chamber, filters, and chlorinating equipment. Chlorine was not employed during the greater part of the first year of operation, excess lime treatment effectively maintaining a sterile effluent. As lime dosage was decreased, after much of material deposited in distribution system through use of raw water had been removed by flushing, chlorination became necessary and is now part of regular treatment. Hardness is reduced from 300 to 100 p.p.m. Secondary application of soda ash, to correct alkalinity depletion by excess lime treatment, has eliminated red water complaints. During first winter's operation, depth of sand in filters increased from 30 to 35 inches due to retention of calcium carbonate, probably due to retardation of carbonate reactions by low temperatures. Reducing depth of sand to 26 inches and increasing wash rate from 23 to about 35 inches vertical rise per minute corrected condition to certain extent, but did not eliminate necessity of screening sand again this spring. During 1930, percentage of wash water used was 1.19, and during 1931,

0.99. Large quantities of concentrated brine discharged by chemical company, 30 miles upstream, seriously interfered with operation. Relief has now been obtained by means of dam and control gates constructed by the chemical company across water plant intake canal: brine is discharged over 24-hour period, or less, and gates are closed while brine is passing the intake. Powdered activated carbon has been used since March, 1931, for taste removal. In July of that year, dosage was increased from 14 to 50 pounds per million gallons. Carbon is fed from standard dry-feed machines into either settling basin or filter influent. Quotations on 13 grades of carbon ranged from $50 to $118 per ton. Brands of carbon which have been more or less successfully used at water purification plants were found to give negative values when tested by method devised by BAYLIS. Adoption of metering has greatly reduced water consumption. Pumpage figures for July, 1930, 1931, and 1932 were, respectively, 584, 375, and 301 million gallons per day.—*R. E. Thompson.*

New Water Supply Serves Pennsylvania Coal Towns. Eng. News-Rec., **110**: 50–2, January 12, 1933. New 10-m.g.d. supply system of Panther Valley Water Co. is nearing completion. Supply will be impounded by rolled-fill dam on Still Creek and conveyed through 30.5-inch welded steel pipe line, 10 miles in length, to distribution systems serving several coal mining towns and a number of collieries. Pipe was lined centrifugally with bitumastic enamel. Lining procedure is outlined.—*R. E. Thompson.*

Cement-Asbestos Pipe Used for Water Line. Eng. News-Rec., **110**: 55–6, January 12, 1933. Cement-asbestos pipe was recently employed in completing auxiliary feeder in Winnipeg. Installation consists of 814 feet of 18-inch and 442 feet of 14-inch pipe, rated at head of 300 feet. Wall thicknesses were 1.62 and 1.26 inches, and weights per foot 90.7 and 54.5 pounds, respectively. Pipe is composed of asbestos fiber and portland cement combined under pressure, has dense homogeneous structure, and is unusually resistant to destructive forces. It is claimed by manufacturer that carrying capacity is higher than that of clean metal pipe. Pipe is made with straight spigot ends and joints are made with sleeves of same material and rubber rings. Details of joint are included. Labor cost was higher than usual in vicinity, but it is believed that, with additional experience, cost can be reduced. Disadvantages are size of bell holes and extra width of trench required. No leakage was observed when tests were made. Soil in Winnipeg is highly corrosive.—*R. E. Thompson.*

Ancient American System for Collecting Water Supply. J. B. HAMILTON. Eng. News-Rec., **110**: 225, February 16, 1933. What is thought to be possibly the oldest water-collecting system in United States is briefly described. It consists of groove about 2 inches wide, 1 inch deep, and 8 feet long in face of cliff near Pueblo Bonito, Chaca Canyon National Monument. Recess, apparently natural, below end of groove could have been used to hold vessel to catch discharge. Near-by ruins are 1000 years old according to dates established by tree rings.—*R. E. Thompson.*

Tank Piers Carried 50 Feet to Rock Through Old Coal Mine. Eng. News-Rec., **110**: 190, February 9, 1933. Brief description of unusual foundation procedure made necessary in erecting 800,000-gallon elevated steel tank for Zanesville, Ohio, owing to partly worked coal mine underlying site.—*R. E. Thompson.*

Shell of Concrete Standpipe Pre-Stressed by Steel Hoops. J. A. JENSEN. Eng. News-Rec., **110**: 224–5, February 16, 1933. Illustrated description of 1.5-million gallon reinforced concrete water tower recently constructed in residential district of Minneapolis, Minnesota. Tank was designed on Hewett system, which involves pre-stressing of concrete with circumferential steel. Plainness of concrete was avoided by widening base, introducing broad pilasters ornamented with figures at intermediate height and spread eagles at top, and by covering structure with dome with tapered ribs terminating at louvered cupola surmounted by beacon light for aviators. Structure has internal diameter of 58.25 feet and height from floor to base of beacon light is 118.25 feet. Depth of water to overflow is 75 feet.—*R. E. Thompson.*

Percolation Slope in Dams Measured by New Device. L. L. MEYER. Eng. News-Rec., **110**: 109–110, January 26, 1933. Description of novel device installed by Los Angeles Water Department in reconstructed Chatsworth Dam No. 2 to determine ground water level, or percolation gradient, in any part of rolled-earth structure, which consists of lines of tubing from well points to recording box on dam crest where interior water levels are determined by simple manometer readings. Air is applied to tube with tire pump, pressure increasing until water reaches bottom of tube, after which pressure will not increase with further air application. Pressure thus indicated is difference in pressure between bottom of tube (elevation known) and water surface standing in pipe (elevation desired).—*R. A. Thompson.*

Rapid Progress Being Made on Vancouver Water Tunnel. Eng. News-Rec., **110**: 56, January 12, 1933. Pressure tunnel being constructed under First Narrows at entrance to Vancouver Harbor by Greater Vancouver Water District to replace present system of submerged mains is scheduled for completion in June. Two shafts, 400 feet deep, and 3104 feet of 7.5-foot tunnel are principal features of development, which is designed for ultimate Capilano Creek supply of 200 million gallons per day. Upper 160 feet of shafts will be lined with steel plates and 4 inches of concrete. Remainder of shafts and tunnel are being lined with steel-cylinder-reinforced concrete pipe. Pipe sections are lined centrifugally with 2 inches of concrete and surrounded by 1, 2, or 3 rows of spiral reinforcing rods. After placing, sections are welded to form continuous pipe, concrete is placed pneumatically between pipe and rock, and in addition grouting is carried on outside pipe. Tunneling work has been completed and half of lining has been placed.—*R. E. Thompson.*

Federal Court Says 5.17 Per Cent Return Not Confiscatory To-day. Eng. News-Rec., **110**: 131, January 26, 1933. In decision handed down on January 12 by three federal judges, it was held that rates prescribed by Illinois Com-

merce Commission for Kankakee Water Company, which would yield probable return of 5.17 percent on investment, were not confiscatory. Decision denied company the right to charge rates higher than that set by commission.—*R. E. Thompson.*

Welded Alloy Bucket Lips Cut Trenching Costs. GEORGE SYKES. Eng. News-Rec., **110**: 215, February 16, 1933. Considerable saving was effected on trenching for 28-mile, 36-inch main from Lincoln to Ashland, Nebraska, by welding wear-resisting alloy edges to worn excavator bucket teeth. Cost of hard-surfacing teeth is but fraction of cost of new ones.—*R. E. Thompson.*

The Microanalysis of Drinking and Utility Water with the Aid of the Pulfrich Photometer. CARL URBACH. Mikrochemie, **5**: 37–60, 1932. From Chem. Abst., **26**: 4116, August 10, 1932. To detect nitrite in water, RIEGLER's naphthol reagent is most satisfactory. Directions are given for preparing reagent and carrying out test. Table is given for interpreting readings of instrument. Treat 10 cc. of water with 0.5 cc. of reagent and 2 drops concentrated hydrochloric acid, shake, add 0.5 cc. concentrated ammonium hydroxide, and examine clear, red solution in photometer. Method is rapid and requires no comparison solutions. Instrument may also be employed in determining ammonia by Nesslerization.—*R. E. Thompson.*

Colloidal Conditioning of Boiler Feed Water. O. E. BLINCOW. J. Inst. Brewing, **37**: 603–7, 1931. From Chem. Abst., **26**: 4116, August 10, 1932. Calcium and magnesium salts may be co-precipitated with added colloid in condition which does not result in scale. Process does not introduce equivalent quantities of sodium salts. Tendency to prime and risk of caustic embrittlement are also reduced. Materials may be incorporated with colloid to neutralize acidity and combine with dissolved oxygen.—*R. E. Thompson.*

Ammonia-Chlorine Treatment at Okmulgee, Oklahoma. ERNEST SKINNER. Okla. A. and M. College, Eng. Expt. Sta., Pub. **15**: 63–5, 1932. From Chem. Abst., **26**: 4117, August 10, 1932. Chloramine reduced algae, aftergrowths, taste and odors, increased residual chlorine, and reduced costs $1.20 per million gallons.—*R. E. Thompson.*

Protecting Pipe Conduits Against Corrosion. N. HAAVARDSHOLM. Tek. Ukeblad **78**: 63–6, 1931. From Chem. Abst., **26**: 4117, August 10, 1932. Description of apparatus for deactivating water by addition of milk of lime (system Riwag) recently installed at Stavanger, Norway. Calculations of outlay and expected savings included.—*R. E. Thompson.*

Oil Removal from Condensate on a Process Plant. CHARLES T. MAIN. Steam Eng., **1**: 334–5, 1932. From Chem. Abst., **26**: 4117, August 10, 1932. Oil in condensate is finely divided and emulsified and cannot be removed by filter unless gelatinous substance such as aluminum hydroxide is precipitated during filtration. Alum and soda ash (to neutralize sulfuric acid) are added for this purpose. Cleaned water is deaërated and treated with phosphate and sodium hydroxide.—*R. E. Thompson.*

Investigation of the Occurrence of Iodine in Waters and Coal and Some Experiments on the Use of Iodine as Fertilizer. R. WACHE. Mitt. Lab. Preuss. Geol. Landesanstalt, **1931**: 13, 43–52; Chem. Zentr., **1931, II**: 981. From Chem. Abst., **26**: 4125, August 10, 1932. Author found from 1.3 to 11.2 γ/liter of iodine in various river and well waters using FELLENBERG method (cf. C. A. **26**: 4154). No regular relation to such factors as iron or salt content, or rocks and minerals, or large cities through which water flowed, could be established. Filtration reduced iodine content about 13 percent.—*R. E. Thompson.*

A. G. A. Studies of Coatings for Pipe Lines. SCOTT EWING. Am. Gas Assoc., Proc., **13**: 774–804, 1931. From Chem. Abst., **26**: 4160, August 10, 1932. In investigation of coatings for pipe lines by the Association and Bureau of Standards, 42 types of coatings were buried for periods of from 9 months to 17 years in 14 selected locations. Coatings which develop numerous pinholes in short period are usually thin: they have temporary value, but will not increase ultimate life of line and are useless in poorly drained soils. Coatings which show limited number of pinholes after short period are usually thick. Thick coatings do not prevent superficial rusting but appreciable pitting is apparently inhibited. Pinholes under thick coatings are primarily due to mechanical action of soil. Organic reinforcing materials are decomposed by bacteria and fungi to degree dependent upon nature of soil. Unvulcanized rubber appears to be unsuitable owing to plastic flow. Pinhole test appears to be best for determining condition of coating.—*R. E. Thompson.*

The Dichotomous Stratification of Hydrogen-Ion Concentration of Some Japanese Lake Waters. SHINKICHI YOSHIMURA. Japan. J. Geol., **9**: 155-85, 1932; cf. C. A., **25**: 4747, 5482. From Chem. Abst., **26**: 4216, August 20, 1932. Study is made of physico-chemical conditions of lakes in which abnormal stratification of pH occurs. In most cases abnormality consists of decrease in pH in upper zone of hypolimnium and increase in lower. This may be due to lack of dissolved oxygen in lower layers. Minimum pH always occurs at level directly above anaërobic strata. Alkalinity in hypolimnium is several times that in epilimnium. Reduction takes place in anaërobic zones. Iron and manganese dissolve in water as ferrous and manganous carbonates, combining with carbon dioxide from bottom deposits, and are precipitated as hydroxides during circulation period. Unusually high ratio between alkalinities at different depths may be due to softness of water and consequent lack of buffer action. Numerous references.—*R. E. Thompson.*

Tests on Practical Comparison of Corrosion of Metals in Water and the Activity of Certain Waters. KLAS SONDÉN. Ing. Vetenskaps Akad. Handl., **110**: 3–24, 1931; cf. Svenska Kommunaltekniska Föreningen, 1928. From Chem. Abst., **26**: 4290, August 20, 1932. In previous tests, it was very difficult to control all conditions, so as to make results comparable. New methods and apparatus depend upon all test samples having same size and form, and large number of samples are tested simultaneously. Corrosions of various metals by hot and cold water, with and without dissolved oxygen and carbon dioxide, are shown.—*R. E. Thompson.*

The Influence of the Hydroxyl Ion on the Germicidal Action of Chlorine in Dilute Solution. W. L. MALLMANN and OSCAR SCHALM. Mich. Eng. Expt. Sta., Bull. **44**: 3–16, 1932. From Chem. Abst., **26**: 4355, August 20, 1932. Increasing pH values above 7.0 caused decreased germicidal action of dilute solutions of chlorine in water up to point where alkalinity itself became germicidal (pH 13).—*R. E. Thompson.*

Internal Corrosion in Zinc-Coated Water Piping. J. FRIEDLI. Schweiz. Ver. Gas-Wasserfach. Monats-Bull., **12**: 65–73, 1932. From Chem. Abst., **26**: 4292, August 20, 1932. Fire coating is compared with electrolytic coating. Latter is often inferior; marked examples of attack are given. Particular attention is paid to thickness of coating, which, in grams of zinc per square meter of surface, should be at least 300, but is frequently less for electrically-coated pipe. Fire-coated pipe is much better in that respect and was found to stand up without inspection; coatings of as much as 1200 grams per square meter were found.—*R. E. Thompson.*

Rapid Filtration of Agar-Agar Media. E. MASERA. Boll. soc. intern. microbiol., Sez. ital., **4**: 126–7, 1932. (In French.) From Chem. Abst., **26**: 4354, August 20, 1932. Filtration under pressure in autoclave is described.—*R. E. Thompson.*

The Raising of the Rate of Filtration Through a Slow Sand Filter. SARDJITO and W. A. L. BAKHOVEN. Mededeel. Dienst Volksgezondheid Nederland.-Indie, **21**: 1–6, 1932. From Chem. Abst., **26**: 4395, August 20, 1932. Increasing rate of filtration at Makassar from 10 to 19.2 cm. per hour did not reduce efficiency, or period of serviceability; but more sand had to be removed when filters were cleaned.—*R. E. Thompson.*

Effect of Salt Concentration on Chloramine Disinfection of Water. ROY E. KING and O. M. SMITH. Okla. A. and College, Eng. Exp. Sta., Pub. **15**: 66–7, 1932. From Chem. Abst., **26**: 4395, August 20, 1932. Salt concentrations up to 500 p.p.m. are without effect: amounts up to 2000 p.p.m. lower the efficiency. —*R. E. Thompson.*

Examination of Tap Water for Small Quantities of Copper and Lead. N. SCHOORL. Z. anal. Chem., **88**: 325–36; Chem. Weekblad, **29**: 338–43, 1932. From Chem. Abst., **26**: 4661, September 10, 1932. Details given for colorimetric determination of small amounts of copper and lead as sulfides, and of copper as ferrocyanide.—*R. E. Thompson.*

Thermal Conductivity of Crystalline Boiler Scale. P. ZARNITZ. Wärme, **54**: 756–61, 1931; Chimie and industrie, **27**: 1069, 1932. From Chem. Abst., **26**: 4396, August 20, 1932. Author studied conditions of formation of different forms of scale and effects of its crystalline structure on thermal conductivity. During evaporation, local saturation occurs at boiler shell, and there are two possible methods of separation of mineral salts: (1) water in contact with shell evaporates, leaving salts adhering to shell, forming deposit with very fine

particles constituting an amorphous scale; (2) layer of liquid in contact with shell becomes saturated and deposits crystals which grow, thus forming crystalline scale. Thermal conductivity of crystalline scales is considerably higher than that of amorphous scale; it depends on composition of crystals and increases with their size. There is no relationship between density and conductivity of crystalline scale, as had been noted with amorphous scale. Drop in thermal conductivity with rise in temperature increases with size of crystals. Structure of scale is same, whether formed at atmospheric, or higher pressure. Tests showed that formation of scale can be avoided by active circulation of water over perfectly smooth heating surface; salts separate in bottom of drum and can be easily removed.—*R. E. Thompson.*

Experiments with Solutions of Chlorine and Sodium Chloride on Pot Plants. P. R. KRONE and F. F. WEINARD. Am. Soc. Hort. Sci. Proc., **27**: 444–8, 1930; Expt. Sta. Record, **66**: 142. From Chem. Abst., **26**: 4626, September 10, 1932. Observations on seedlings of flowering plants watered with chlorinated water and salt water showed distinct injury with former in concentrations greater than 5 p.p.m. In case of sodium chloride, 100 p.p.m. caused no injury in 1 month, but slight burning of zinnias was noted with 200 p.p.m. Mixture of sodium chloride and sulfate was less harmful than either alone.—*R. E. Thompson.*

Chlorination of Water. JOHN C. BAKER and FRANZ C. SCHMELKES (to Wallace and Tiernan Products, Inc.). U. S. 1,866,524, July 12. From Chem. Abst., **26**: 4662, September 10, 1932. Disagreeable odor and taste from treatment of water with chlorine for sterilization is avoided by adding small proportion of iron salt, such as sulfate, and of ammonia, or ammonium compound, such as sulfate.—*R. E. Thompson.*

Preservation of Samples of Water and of Aqueous Soil Extracts. S. M. DRACHEV and R. A. KALACHNIKOVA. Z. anal. Chem. **87**: 173–9, 1932. From Chem. Abst., **26**: 4896, September 20, 1932. Experiments with chloroform, toluene, thymol, zinc oxide, and mercurous oxide show that none of these is perfect and that slight changes in condition of nitrogen and in sulfate content occur on standing. Changes, however, are much slower than ordinarily thought, and above preservatives all tend to retard them. Mercurous oxide was found most effective.—*R. E. Thompson.*

Poisonous Varnish for Hot Water Storage Tanks. WILH. HUTH. Farbe u. Lack, **1932**: 83–4. From Chem. Abst., **26**: 4967, September 20, 1932. Careful consideration shows that it is impossible for individuals to consume enough hot water to be poisoned by lead salts extracted from interior coatings of storage tanks.—*R. E. Thompson.*

Water Turbidity and Turbidity Measurements. ALFRED DRENNIG. Z. österr. Ver. Gas-Wasserfach., **72**: 133–8, 1932. From Chem. Abst., **26**: 6039, November 20, 1932. Apparatus for measuring turbidity is described, in which light from standard source passes through circulating column of water under

test, transmitted portion falling upon photoelectric cell. Current from latter causes condenser to become charged: condenser automatically discharges at given potential density and cycle is repeated. Time required to charge condenser is measure of turbidity and is recorded on revolving drum.—*R. E. Thompson.*

Formate Ricinoleate Broth: a New Medium for the Detection of Colon Organisms in Water and Milk. C. N. STARK and C. W. ENGLAND. J. Bact., 29: 1, 26–7, 1935. New medium, consisting of 0.5 per cent peptone, 0.5 per cent lactose, 0.5 per cent sodium formate, and 0.1 per cent sodium ricinoleate, accelerated growth in *Escherichia-Aërobacter* group, caused total population of bacteria to be much larger, and greatly increased total gas production. Due partly to its buffering effect, sodium formate accelerates growth and increases bacterial population. Gas production is increased because *Escherichia-Aërobacter* and *Salmonella* groups can produce gas from formic acid, in absence of any other fermentable substance.—*Selma Gottlieb.*

Bacterial Contents of Chlorinated Swimming Pools During Periods of Use. W. L. MALLMANN. J. Bact., 29: 1, 26, 1935. Using sodium-thiosulfate-treated sample bottle, large numbers of bacteria were found present during periods of heavy swimming loads, as shown by 37°C. total counts and *B. coli* and streptococcus indices. By regulating bathing load, bacterial contents of pools could be held within present standards recommended by American Public Health Association Committee on Bathing Places. In general, chlorine residual should be held at 0.4 p.p.m. Critical period of pollution was found to occur shortly after bathers entered water.—*Selma Gottlieb.*

A Glucose Iron Bismuth Sulphite Medium for Quantitative Estimation of E. typhi in Water, Sewage, and Sewage Sludges. C. C. RUCHHOFT. J. Bact., 29: 1, 25–6, 1935. Endo, E. M. B., and brilliant green agar were found too inhibitive of *E. typhi* and not sufficiently inhibitive of other sewage organisms when compared with nutrient agar. A modified WILSON and BLAIR'S medium was found 92 per cent productive of *E. typhi* and only 1.8 per cent productive of other sludge bacteria, as compared with nutrient agar. Detailed directions are given for preparing the medium. *E. tyhpi* has been isolated with it from Chicago activated sludge, and death rate curves of *E. tyhpi* in stored activated sludge have been obtained. *E. typhi* has been isolated from sludge samples with this medium when for each *E. typhi* there were present 1000 other members of coli-aërogenes group and 25,000 other bacteria.—*Selma Gottlieb.*

Effect of Temperature and Boric Acid on Gas Production in the Colon Group. REESE VAUGHN and MAX LEVINE. J. Bact., 29: 1, 24–5, 1935. Using buffered peptone lactose broth, containing 10 cc. of ANDRADE'S indicator and 3.25 grams of boric acid per liter, 98 per cent of 148 strains of *Escherichia* grew luxuriantly after 48 hours at 43° to 44°C., whereas only 3.3 per cent of 181 strains of *Aërobacter* and none of 53 strains of *Citrobacter* showed growth. Evidence of inhibition of gas production was found with some strains which

grew. Of the Escherichia cultures, 95.3 per cent produced gas, whereas none of the *Aërobacter* or *Citrobacter* strains did so, after 48 hours at 43–44°C.—*Selma Gottlieb.*

The Relative Productivity of Various Fermentation Media as Compared with Standard Lactose Broth by the Butterfield-Hoskins Method. R. E. NOBLE. J. Bact., 29: 1, 23–4, 1935. Fuchsin lactose, formate ricinoleate, crystal violet, brilliant green lactose bile, and Difco lactose broths were tested separately, in parallel with standard lactose broth, by (1) conventional inoculation from 61 coli-aërogenes stock strains, and (2) the BUTTERFIELD method. Single stock strain of *B. coli* was used as test organism and Most Probable Number, to express results. Uniformly, all trial and standard media produced gas within 42 hours of incubation at 37°C. after conventional inoculation of 61 stock strains, in mass amounts. However with BUTTERFIELD-HOSKINS method, using limited number of organisms, only formate ricinoleate broth and Difco lactose broth met requirements, as compared with standard medium. —*Selma Gottlieb.*

The Fermentation of Cellobiose by Bacteria. RALPH P. TITTSLER and LESLIE A. SANDHOLZER. J. Bact., 29: 1, 20–21, 1935. Ability to ferment cellobiose was determined for 755 cultures, representing approximately 50 species in various genera. Acid and gas were produced by the *Aërobacter* cultures and most *Escherichia-Aërobacter* "intermediates" but not by typical *Escherichia* cultures. All strains of *Eb. typhi*, *Sal. aërtrycke* and *Sal. schottmuelleri* and 17 of 19 strains of *Sal. enteritidis* attacked cellobiose after 4 to 10 days; whereas all strains of *Sal. suipestifer*, *Sal. paratyphi*, *Sal. gallinarum*, *Sal. pullorum*, *Sal. morgani*, and of dysentery bacilli failed to do so within 14 days.—*Selma Gottlieb.*

Some Quantitative Relationships in Bacterial Population Cycles. M. W. JENNISON and J. W. M. BUNKER. J. Bact., 29: 1, 19–20, 1935. Growth cycles of four common saprophytes, including *B. coli* (two strains) and *B. aërogenes* were determined. While maximum *rates* of reproduction of different organisms at same temperature were nearly equal, maximum *numbers* of cells showed wide variation. For any one species maxima reached were fairly consistent, irrespective of temperature. At each of four temperatures used, ratio of *rate* of reproduction to *length* of logarithmic growth phase was quite consistent. Considered as typical population curves, these cycles were, as a rule, very asymmetric.—*Selma Gottlieb.*

Germicidal Properties of Water Containing Minute Amounts of Silver. ROBERT P. MYERS and J. C. MAUER. J. Bact., 29: 1, 7–8, 1935. Results on dilute silver solutions prepared from fused silver chloride, silver nitrate, silver acetate, colloidal silver, and Katadyn silver indicate that 0.10 to 0.15 p.p.m. of silver ion is sufficient to destroy 250,000 *B. coli* per cc. of river, or neutral distilled water, within 2½ hours. *Staph. aureus* appeared only slightly more resistant.—*Selma Gottlieb.*

Determination of Chloride. JOHN R. CALDWELL and HARVEY V. MOYER. Ind. Eng. Chem., Anal. Ed., 7: 1, 38–9, 1935. To eliminate necessity of filtering in chloride determination by VOLHARD method, one cc. of nitro-benzene per 0.05 gm. of chloride is added before titration. Nitrobenzene forms insoluble layer over precipitate, fading endpoint is prevented, and results are accurate. —*Selma Gottlieb.*

Determination of Fluorine in Drinking Water. H. V. SMITH. Ind. Eng. Chem., Anal. Ed., 7: 1, 23–5, 1935. Methods of WILLARD and WINTER, FOSTER, SANCHIS and FAIRCHILD were compared, using 44 water samples. Results by first three methods agreed closely, but results by FAIRCHILD method averaged two or three times as high. From results by first three methods, all waters containing over 1.0 p.p.m. of fluorine were definitely associated with mottled enamel, but none containing less than 0.8 p.p.m. was known to cause it.—*Selma Gottlieb.*

The Removal of Lead and Copper from Drinking Water. KARL HÖLL. Archiv für Hygiene und Bakteriologie, **113**: 5, 216, February, 1935. Four gallons lead-free water may be obtained by single filtration through cotton wool plug of fist-size, if original lead content does not exceed 1 p.p.m.; and through filter paper mass, if original content does not exceed 2.5 p.p.m. Most suitable method for household purposes is filtration with activated carbon; of which from 0.6 to 2.0 grams are added to 1 liter, before pouring off through a folded filter containing 5 grams of carbon. Berkefeld filter candles, as used for sterilization, yield a definite quantity of lead-free filtrate, decreasing with increasing lead content of water; it is essential to ascertain when regeneration becomes necessary. These filter candles are unfit for water with over 200 p.p.m. of chloride chlorine. For larger installations, filtration through marble will not secure continuous lead removal, once period of initial activity is over; as especially effective, however, filtration through burnt granular magnesite is recommended; with which tests indicated complete removal of added lead in concentrations up to 50 p.p.m. from distilled water containing 200 p.p.m. of carbon dioxide. This material proved effective for copper removal also.—*Manz.*

Drinking Water Supply of the Island of Heligoland and the Occurrence of Lead Therein. KARL HÖLL. Archiv für Hygiene, **113**: 5, 283, February 1935. Due to unfavorable conditions in underlying sandstone rock, residents are exclusively supplied by rain water collected in cement-lined masonry cisterns, on ground or basement level, with capacities ranging from 2600 to 8000 gallons. For distribution, hitherto lead pipe and in some cases even lead-lined water tanks have been used without filtration or other purification. Local conditions cause exceptional pollution owing to washing of ash and soot from the roofs and to leaching of bird droppings and even of carcasses. Cistern water is often yellow and permanganate consumption, from 40 to 60 p.p.m. (i.e. oxygen consumption from 10 to 15 p.p.m.). Due to high salt content of rain water and copious showers of sea spray during stormy weather, chloride content averages 50 p.p.m. and attains maximum values of 300 p.p.m.; hardness

and free carbon dioxide vary to similar extent. Owing to repeated occurrence of symptoms reminiscent of chronic plumbism, comprehensive study was undertaken. Plumbosolvency is governed by relation between free and combined carbon dioxide, being less than 0.5 p.p.m. in waters with free carbon dioxide considerably less than one-half of the combined; but exceeding 1 p.p.m. when content of free carbon dioxide approaches one-half of that of combined. Neither the low oxygen content of 4 p.p.m., nor the chloride concentration present exerts any apparent influence. Out of a total of 233 samples examined, 82 showed no detectable lead content; 60 showed from 0.1 to 0.3 p.p.m.; and 91, more than 0.3 p.p.m., with an average of 0.9 p.p.m.; while 0.67 p.p.m. was the average for the whole 233. These findings are definitely lower than the corresponding data from previous plumbism epidemics at Dessau and Leipzig; it is therefore concluded that continuous, or even lifelong, ingestion of water with small amounts of lead, not exceeding 0.2 p.p.m., may not be injurious to health. As a prophylactic measure, however, removal of all lead equipment was ordered.—*Manz.*

Methods for Chlorine Consumed Test in Water. H. IVEKOVIC. Arhiv za Hemiju i Farmaciju, **8**: 5/6, 185, 1934. Distinction is made between chlorine absorption value according to ELMANOVITCH and ZALESKI, or FROBOESE, and chlorine binding power according to BRUNS-OLSZEWSKI. For purpose of comparison with permanganate demand, only chlorine absorption is considered and not other procedures using much lower chlorine concentration. For determination of chlorine absorption, modification of KAESS is recommended, as furnishing good and concordant results. Mixture of 100 cc. sample and 20 cc. N/50 sodium hypochlorite solution, after 30 minutes in boiling water bath, is cooled, treated with from 0.5 to 1 gram potassium iodide and 5 cc. 1:1 hydrochloric acid and titrated with N/50 sodium thiosulfate solution. High ratio of chlorine absorption to permanganate demand does not necessarily indicate fecal pollution, but only presence of albuminoid matter, which may be of fecal, or of vegetable origin. Neither may it always be interpreted as an index of decomposed albumen. It is only when considered in connection with other data of the analysis that chlorine absorption test allows valuable conclusions to be formed regarding the organic matter present.—*Manz.*

Determination of Oxygen Consumed in Natural Water and Sewage. HELLMUTH STAMM. Angewandte Chemie, **48**: 9, 150, March, 1935. Proposed method is based upon the principle of limiting the reduction of alkaline permanganate used to the first stage only, namely to manganate, latter being converted into insoluble barium manganate, and thus withdrawn from reaction. Owing to high velocity of the reaction, test may be completed rapidly, without heating, in the field. To 100 cc. sample, 5 grams barium chloride, from 3 to 3.5 grams sodium hydroxide, and 20 cc. N/10 potassium permanganate solution are added; after 10 minutes, sample is titrated with N/10 sodium formate, from 15 to 20 drops 1 per cent nickel nitrate solution being added in order to hasten the final decoloration. Tests on Saale water gave somewhat higher results than usual procedures.—*Manz.*

The Tyndall Effect for Examination of Springs. MAXIMILIAN KNORR. Archiv für Hygiene und Bakteriologie, **113**: 5, 274, February, 1935. Bacteriological results and local inspection suggested that a certain spring supply was subject to contamination by surface run-off. As tests with salt, or fluoresceine, were inadmissible, changes in water quality following after rain were proved by increasing intensity of Tyndall turbidity, believed to be partly due to traces of iron.—*Manz.*

Analysis of Thick Arch Dams, Including Abutment Yield. P. CRAVITZ. Proceedings, American Society of Civil Engineers, **61**: 1, 43, January 1935. Graphical solution of stresses in a circular arch, under influence of normal loads as presented on an arch dam. Curves developed differ from those previously published, in that they include the effect of abutment yield. A finite value of POISSON'S number is used, rather than the infinite value previously assumed for simplification. Derivation of final stress equations is traced, because it combines advances made by other investigators. Forms of the equations are greatly altered, in order to facilitate computations and plotting of graphs. Some examination is made to determine the quantitative effect of deviations in the modulus of elasticity of rock, and in POISSON'S number from the values necessarily assumed. Example of use of the curves is appended.—*H. E. Babbitt.*

Water Supply Engineering. Progress Report of the Committee of The Sanitary Engineering Division. Proceedings, American Society of Civil Engineers, **60**: 10, 1463, December 1934. First division of the report is devoted to activities of organizations interested in water supply engineering. List of 21 of the better-known national and regional associations is given, with statement of their activities in water works field. Suggestions for activities of the Am. Society of Civil Engineers in connection with water works problems are given at some length. Particular stress is laid upon the value of research, six important problems of which are described. The second division of the report is entitled: Status of The Art of Water Supply Engineering. Examples are given of new types of construction, including design, fabrication, or processes. Developments in available materials are pointed out, such as the trend away from the so-called standard specifications for Portland cement. The advanced engineer is now specifying cement to meet his particular needs. New methods are described for placing and cooling of mass concrete; for construction of earth dams; and for fabrication and coating of steel and cast-iron pipe. In the field of water purification, the usefulness of activated carbon in taste and odor control is discussed, together with such subjects as the application of chlorine to water, baffles for sedimentation basins, and the sealing of wells against salt. Recent advances in improved pumps and prime movers are emphasized and stress is placed upon increased use of electric motors and steam turbines.—*H. E. Babbitt.*

Model Research on Spillway Crests. H. ROUSE and L. REID. Civil Engineering, **5**: 1, 10, January, 1935. A study of pressure distribution and of discharge as a function of crest design. In design of spillways, there are three

essential requirements to be met: (1) maximum expected discharge must be passed with least possible increase in elevation of reservoir surface; (2) structure must be not only stable under all conditions, but also of economical construction; and (3) hydraulic characteristic of flow over crest should be known more definitely than is now possible on the basis of unverified design. American engineers, as a rule, base the curve of spillway crest on the profile of the fully ventilated nappe of water flowing over sharp-crested weir. Such methods are founded on the belief that presence of a solid structure below the nappe, and in immediate contact with it, will not appreciably affect the course of freely falling particles of water. The three forces affecting flow of water over a weir are gravitation, viscosity, and surface tension. Although first is the primary force, the others play significant rôles in altering magnitude of discharge coefficient. Measurements have been made of flows over various weirs, coefficients for which are given. In some cases where the common guess might have been in neighborhood of 3.8, figure has been found to be nearer 3.4. Although such experiments on small models as are here discussed cannot be expected to give accurate quantitative results for operation of large-scale spillways, qualitatively their indications are of direct importance in design.—*H. E. Babbitt.*

Geology of Boulder and Norris Dam Sites. C. P. BERKELEY. Civil Engineering, 5: 1, 25, January, 1935. Discussion of the major features. It is not practicable to generalize on dam sites, each of which is a new problem and presents features differentiating it from all others. Necessity for careful geological study in every case is clearly demonstrated. In this particular study of two of the country's greatest problems of their kind, interesting geological facts are developed.—*H. E. Babbitt.*

Foundation Conditions for Grand Coulee and Bonneville Projects. C. P. BERKELEY. Civil Engineering, 5: 2, 67, February, 1935. Continuation of the interesting geological discussion initiated in preceding issue (see preceding Abstract) points out unusual and unique foundation conditions on these great Columbia River development projects. Explanations of the formations encountered help one to visualize the difficulties confronting designers and builders of dams.—*H. E. Babbitt.*

Colorado River Aqueduct Makes Rapid Progress. A Symposium. Civil Engineering, 5: 2, 72, February, 1935. History, design, and construction of the huge water supply project for Southern California. **Organization, Location, and Financing.** F. E. WEYMOUTH. **Design of Hydraulic Structures.** JULIAN HINDS. **Methods and Progress of Tunnel Construction.** J. L. BURKHOLDER. **Tunnel Contractors' Plant and Equipment.** J. C. AGNEW. Unparalleled progress is being made on construction of aqueduct which is to conduct 1,500 cubic feet per second from the Colorado River for 240 miles, across deserts and through mountains, to Los Angeles. Thirteen contractors and forces of Metropolitan Water District of Southern California are digging the tunnels at rate of four miles a month, and apparently all existing records have been broken for rate of advance of a single heading. More than 60 per-

cent of total 91-mile tunnel mileage had been excavated. Although main effort has been concentrated on the tunneling, attention is now being paid to construction of diversion dam across the Colorado River at Parker, the cut-and-cover conduit, and the 150 inverted siphons. First work on the tunnels in distribution system has also been undertaken. The articles are profusely illustrated with maps, diagrams, and photographs.—*H. E. Babbitt.*

Changes in Lake Levels in Great Basin Area. S. T. HARDING. Civil Engineering, **5**: 2, 87, February, 1935. Three hundred years of record are studied to determine trend in available water supply. Fluctuations in level of the enclosed lakes of the Great Basin reflect the water supply conditions on their drainage areas. Records of such fluctuations include both direct measurement of elevation of lake surface and indirect records obtained from trees growing around margins of lakes. While results for different lakes are found to vary, they are reasonably consistent in their general trends. Among more interesting conclusions are: (1) that within past 30 to 70 years, Great Basin area has apparently experienced its most moist period of the last 300 years; (2) the period of deficient water supply since about 1917 has not resulted in as great an accumulated deficiency as has occurred in the past; (3) this record does not support the conclusion that the climate is becoming progressively drier; and (4) no cyclic variations have been found which justify projection into the future.—*H. E. Babbitt.*

The Hydraulic Jump in Terms of Dynamic Similarity. B. A. BAKHMETEFF and A. E. MATZKE. Pro. Am. Soc. Civil Engineers, **61**: 2, 145, February, 1935. It is evident that in the design of hydraulic structures in which the jump occurs, knowledge of its longitudinal elements is no less important than that of its vertical elements, upon the study of which attention has until now been chiefly concentrated. The longitudinal elements of the jump have been studied in the Fluid Mechanics Laboratory of Columbia University during 1932–1933. A particular feature of the work was, that in interpreting and systematizing results obtained, recourse was taken to principle of dynamic similarity and final data were presented in generalized dimensionless forms. Results obtained seem to confirm the usefulness of general methods applied and are claimed to have given the first comprehensive picture of the longitudinal features of the hydraulic jump in general.—*H. E. Babbitt.*

Frictional Resistance in Artificially Roughened Pipes. V. L. STREETER. Proceedings American Society of Civil Engineers, **61**: 2, 163, February, 1935. Although laws of flow of fluids in smooth pipes are well understood, problem of estimating friction losses in old pipes has been inadequately solved. This paper presents results of a study of frictional resistance in artificially roughened pipes. With water as the fluid, loss of head was investigated for roughened, 2-inch, brass pipe, the REYNOLDS numbers ranging from 20,000 to 1,250,000. Roughness elements consisted of grooves cut spirally into the pipe. Degree of roughness was changed by changing the depth, shape, and number of grooves per inch. Loss of head due to friction was found to increase

with increase in depth of grooves, provided same general shape was maintained. Shape of grooves, however, seems to have almost as much effect on loss of head as their depth. Experiments also indicate that roughness elements may be placed so close together that loss of head is reduced. Work of previous investigators is summarized briefly; the apparatus, preliminary experiments, experiments for obtaining data, and computations are described; and results of this investigation are discussed.—*H. E. Babbitt.*

Stabilizing Constructed Masonry Dams by Means of Cement Injections. D. W. COLE. Proceedings American Society of Civil Engineers, **61**: 2, 187, February, 1935. Some masonry dams, located in India, about 60 miles inland from Bombay, were constructed of rubble masonry, faced with ashlar, and have been in use since 1917. In recent years, increase of seepage through and under dams gave rise to some apprehension as to their continued stability and as remedy, cement injections were prescribed. Paper describes methods and results of borings and injection of 64,000 barrels of Portland cement into 380,000 linear feet of drill holes in three dams, working under full reservoir conditions. Some of the observations give pertinent information on the moot questions of uplift, sliding influences, and the reliance to be placed on the usual assumption of monolithic structure in dam design.—*H. E. Babbitt.*

Water Rights of Riparian Owners. LEO T. PARKER. Water Works Eng., **87**: 15, 821, July 25, 1934. Riparian owner is an owner of land bounded by a water course or lake; courts have decided that riparian owners can divert water only for use on watershed of stream: case is cited where city sold water for industrial purposes to another contiguous city and damages were awarded lower riparian owners for loss of water. If riparian owner proves that another riparian owner is using, or appropriating, water illegally, court will grant an injunction against such use. Case is cited where private water company was permitted to appropriate public lands in developing source of public water supply. Riparian owners are liable in damages if materials stored on banks of stream are carried down stream during floods, if said debris causes damage to lower riparian owners.—*Lewis V. Carpenter.*

Important Water Supply Laws. LEO T. PARKER. Water Works Eng., **87**: 16, 875, August 8, 1934. All water contracts not in accordance with state law are invalid. Case is cited where a water company had agreed, among other things, to provide sufficient pressure for extinguishing fires at any point in the city, and fire during time of low water pressure destroyed valuable property: courts held water company liable in damages for value of property destroyed. State law requiring Hartford, Conn., to serve water to West Hartford at same rates was held valid. Water companies can establish regulations to collect rents and require advance payments; but, unless state law specifically permits, they cannot cut off water supply for non-payment of arrears due from former owner. A water bill is a lien against property, only by state law: city ordinance causing water bill to be a lien against real property is valid only when state law so provides.—*Lewis V. Carpenter.*

Tracing Ground Water Flows. H. O. Williams. Water Works Eng., 87: 17, 920, August 22, 1934. Three or four wells drilled to water-bearing strata are usually desirable. Dyes such as fluorescein, or eosine, are added at bottom of one well by means of a pipe and their subsequent appearance in the other wells detected by noting the color of the water as it is pumped. Fluorescein can be detected at a dilution of one part in two billions; but it is useless in waters which contain free acid. Soluble salts of sodium, calcium, or lithium, are frequently used; but some form of chemical analysis is necessary for their subsequent detection. A third method described uses an electrolyte, such as sal ammoniac, and with proper connections with dry cells, the change in electrical conductivity is noted on a micro-ammeter. Methods described are usually adapted to measuring direction of flow, rather than quantity.—*Lewis V. Carpenter.*

Maintaining Chlorine Gas Masks. James A. Parks. Water Works Eng., 87: 17, 923, August 22, 1934. Canister type gas masks are placed outside of each chlorine station of the Detroit Water System. Every four months they are brought into the laboratory and subjected to rigid inspection. The Burrell Industrial gas mask is used, with "GMB" canister. Life of canister is set at two hours and operator keeps a record of time they are in service.—*Lewis V. Carpenter.*

Effects of Fluorine in Water on Teeth. Frederick S. McKay. Water Works Eng., 87: 17, 925, August 22, 1934. Writer describes in detail experiences at Oakley, Idaho, where municipal water supply had fluorine content of 6 p.p.m. and school children all had mottled enamel. New supply which contains less than 0.5 p.p.m. seems to have no effect on the enamel. From results of these studies it is concluded that fluorine content of 2 p.p.m. will affect the teeth during period of calcification. Table giving fluorine contents of water in a number of towns includes some as high as 16.8 p.p.m. Article traces 92 cases of children in the affected area.—*Lewis V. Carpenter.*

Dubuque Water Plant Prospers. Anon. Water Works Eng., 87: 18, 975, September 5, 1934. Article descriptive of system, which takes water from five artesian wells by air lift. Several distributing reservoirs have been constructed and plant has been able to reduce rates by 10 percent last year.—*Lewis V. Carpenter.*

Painting Fire Hydrants. Nathan N. Wolpert. Water Works Eng., 87: 21, 1154, October 17, 1934. Author outlines necessity for painting hydrants and gives the Maine Utilities Association's recommendations therefor, by which class of fire stream obtainable from hydrant is indicated. Hydrant barrels are painted chrome yellow, for maximum visibility, and caps, either green, orange, or red, depending on pressure available.—*Lewis V. Carpenter.*

Automatic Control Aids Pumping Station Efficiency. James F. Conlan. Water Works Eng., 87: 22, 1212, October 31, 1934. Village of Scarsdale, N. Y., placed in operation a fully automatic pumping station located beneath a

traveled highway. Water is obtained from New York supply and pumped against head of 101 feet. Pressure control panel controls time switch for shutting station down during peak hours and contact-making pressure gages for stopping or starting either pump, so as to maintain proper level in standpipe. Complete protection against overload, under-voltage, or single phase operation, is provided by protective relays in each synchronous motor cubicle. Synchronizing relays are of frequency-responsive type, which automatically remove field excitation in event of synchronizing motor falling out of step, due to voltage disturbances, or other causes, and permit motor to re-synchronize automatically when normal conditions are restored.—*Lewis V. Carpenter.*

Removal of Iron and Manganese from Water. S. P. APPLEBAUM. Water Works Eng., 87: 24, 1312, November 28, 1934. Two general methods for removing iron and manganese from well waters are (1) conversion by oxidation to hydrated higher oxides and (2) base exchange through zeolites, without oxidation. Carbon dioxide, if high, must first be removed. Iron-bearing waters low in pH value do not respond satisfactorily to aëration, as precipitation of iron hydroxide requires a pH value of over 7 and often of over 8. When organic matter is present, special coagulants are required. Excessive aëration frequently interferes with complete removal of iron and manganese. For water low both in iron and in carbon dioxide, add air into suction of pump and filter through sand. For waters high in iron and low in carbon dioxide, add air, pass through coarse contact filter followed by sand filter, or by alkali feed, or aërator (coke tray), before filtration. When iron is present with low organic matter, special chemical treatment is required. Sometimes alum and frequently lime will coagulate iron. At Elizabeth, N. C., treatment included primary coagulation with chlorinated copperas, secondary coagulation with lime and alum, filtration, and adjustment of pH with lime. Later, this was modified by coagulating with lime and potassium permanganate, followed by secondary coagulation with chlorinated copperas. Manganese zeolite (sodium zeolite treated with manganous chloride) has been found completely to remove manganese by base exchange. Sodium zeolite will remove iron by base exchange and has been found to do so even in the "twilight zone."—*Lewis V. Carpenter.*

Liability for Failure of Supply. LEO T. PARKER. Water Works Eng., 87: 24, 1316, November 28, 1934. Under ordinary circumstances, neither municipality, nor water corporation, is liable for failure or neglect to furnish sufficient water for extinguishing a fire, if no contract exists between corporation and property owner, and if no state law renders corporation liable. Author lists a number of states in which above principle has been confirmed in the higher courts. In cases where water company entered into contract to furnish water at a given pressure and failed to do so during a fire, courts have held company liable in damages. Another important law is, that under ordinary circumstances either a water company or a municipality may, without liability, turn off the water when a fire is raging, if it is necessary to do so in order to safeguard the citizens. A company may shut off a water supply, if rules and regulations are disobeyed; but only after serving notice on consumer. Com-

pany cannot shut off water for any debt not directly connected with water service. Court will not render verdict in favor of water company who sues consumer to recover payment for water, when such suit is based on retroactive law, such as an ordinance enacted after the consumer had used the water. Courts usually favor ordinances enacted to enforce payment of water bills.—*Lewis V. Carpenter.*

NEW BOOKS

Elimination of Taste and Odor in Water. JOHN R. BAYLIS. First Edition. McGraw-Hill Book Company, New York and London, 1935. Few subjects in water supply and water purification have attracted as much attention as the newly developed technology for the removal of objectionable tastes and odors. Although the subject is not new, the successes and failures of earlier experimenters have been attacked by American chemists with the aid of more efficient apparatus and materials, so that in about ten years a technique has been evolved for successfully combatting the problem of taste and odor in water supply in the concentrations normally encountered. Educated to expect clarity and purity of water supplies as its due, the American public has of late become more fastidious as to aesthetic quality and the newer methods of taste and odor removal are in the nature of a boon to the much harassed plant superintendent.

Among the experts on water purification in the United States, John R. Baylis is particularly qualified to write a book on the subject of taste and odor removal from public water supplies. In addition to his years of practical experience in the control and operation of the Montebello filter plant at Baltimore, he has occupied a unique position in the field of water purification in this country for a number of years as chemist in charge of the experimental work on water purification carried on by the City of Chicago.

Mr. Baylis' book is by no means a simple outline of methods and applications. It is rather in the nature of a treatise. The author traces the development of each idea from its inception with more than ordinary attention to historical detail. For this reason the book is not easy reading. However, anything lost in this respect is more than made up by its value as a reference work. Throughout its pages it reflects the author's careful and painstaking method of approach to the problems under discussion.

Out of a total of 368 pages in the book, 209 are devoted to the subject of activated carbon. Included in this exhaustive treatment will be found the historical background, the properties of carbon, experimental work, practical application, the use of powdered and granular material, specifications, methods of testing, and many other details. Following the chapters on carbon there are three chapters on the ammonia-chlorine treatment comprising some 50 pages and covering history, application, apparatus, chemistry of the process, its uses and results. The remainder of the book consists of three chapters devoted respectively to the presumably less important processes of super and de-chlorination, aeration, ozone, potassium permanganate, and bleaching clays.

The book is devoted entirely to methods of taste and odor removal as applied at the purification plant. No mention is made of the copper sulphate treatment for algae, which is a preventative measure and is thoroughly described in other books. The author avoids extravagant claims for any of the treatments as will be evident from the statement (page 147): "It may be stated that, with the exception of chlorine, no compound which occurs in water in fairly high concentrations can be removed economically by activated carbon."

Aside from the comprehensive text, the book will be welcomed because it contains the details of numerous experiments carried out at the Chicago experimental station. Much of this material and, in fact, portions of the text have appeared in the current waterworks journals from time to time; but in the book all of this material is correlated to the general purpose of the work.

The publication of Mr. Baylis book had scarcely been announced when the report of the Sub-Committee of the American Water Works Association on Activated Carbon was presented at the Cincinnati Meeting. The Committee favors the use of the modified Fox and Gage method for phenols over the Baylis-Gibbs method recommended by Baylis. With the exception of this test and perhaps a few minor points of difference, it is believed that the Committee report will be found to be in close agreement with the book under discussion.

The general approach of the author has been from a chemical and physical standpoint, but engineers will find much of value for design purposes. The advantages of filtration through granular activated carbon are stressed and typical layouts are discussed. This method has not attained the popularity it deserves on account of the cost of suitable material and the somewhat elaborate installation required for large scale units. No mention is made of the difficulty experienced with fine particles of carbon appearing in the effluent of upward flow granular carbon units.

In considering results obtained by the author in his experiments on samples of water shipped to him from outside plants, it should be remembered that odors may diminish due to storage and that no single sample is representative of the fluctuations in quality experienced during a year's operation. Cost estimates based on such tests are of little value. Of costs in general the author believes it safe to say that "all objectionable tastes and odors can be removed from the water of over 99 percent of the water supplies at a cost which the water works can afford to pay." It is possible that some consumers and waterworks officials might wish to argue this point.

"Elimination of Taste and Odor in Water" is a timely and valuable addition to waterworks literature and it will be useful to all waterworks chemists, operators and engineers. The book is printed on excellent paper and is attractively bound. The illustrations, cuts and drawings, although moderate in number, are clear and interesting. There is an author and subject index.—*George D. Norcom.*